US Army Corps
of Engineers®
Los Angeles District

Little Colorado River

Feasibility Study Report

APPENDIX B

Hydraulics

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LIST OF ACRONYMS

ACE – Annual Chance Exceedance
ADOT – Arizona Department of Transportation
ADWR – Arizona Department of Water Resources
ATR - Agency Technical Review
BNSF – Burlington Northern Santa Fe (Railroad)
CFS – Cubic Feet per Second
CNP – Conditional Non-Exceedance Probability
DQC – District Quality Control
DTM – Digital Terrain Model
EC – Engineering Circular
FEMA – Federal Emergency Management Agency
FIRM – Flood Insurance Rate Map
FIS – Flood Impact Study
FPS – Feet per second
FRRM – Flood Risk Reduction Measure
GDS – Grid Developer System
GIS – Geographic Information System
HEC-FDA – Hydrologic Engineering Center - Flood Damage Reduction Analysis
HEC-GeoRAS - Hydrologic Engineering Center – Geospatial River Analysis System
HEC-RAS – Hydrologic Engineering Center - River Analysis System
I-40 – Interstate 40
LCR – Little Colorado River
NAD – North American Datum
NAVD – North American Vertical Datum
NFIP – National Flood Insurance Program
PDT – Project Delivery Team
RW – Ruby Wash
RWDL – Ruby Wash Diversion Levee
R&U – Risk and Uncertainty
SR 87 – State Route 87
TSP – Tentatively Selected Plan
USACE – United States Army Corps of Engineers, Los Angeles District
USBR – United States Bureau of Reclamation
USGS – United States Geological Survey
WL = Winslow Levee
WSE – Water Surface Elevation
WWTP – Wastewater Treatment Plant

EXECUTIVE SUMMARY

The U.S. Army Corps of Engineers, Los Angeles District (USACE), is currently conducting the Flood Risk Management Feasibility Phase for the Little Colorado River (LCR) at Winslow Study, a cost shared effort between the USACE and the Navajo County Flood Control District.

The purpose of the LCR at Winslow Feasibility Study is to develop and evaluate potential nonstructural and structural engineered solutions to address flooding issues within and near the City of Winslow in Arizona.

In order to determine the water surface elevation on the river side of the existing levee, baseline condition flow breakout analysis for the LCR was conducted using Hydrologic Engineering Center River Analysis System (HEC-RAS). For this model, it was assumed the existing levee does not fail. Water surface profiles were computed for the 50%, 20%, 10%, 4%, 2%, 1%, 0.5%, and 0.2% Annual Chance Exceedance (ACE) floods. Floodplains for the eight frequencies are displayed on Plates 8 to 15, respectively. The 50%, 20%, 10%, and 4% ACE floodplains do not show flooding in the City of Winslow as the existing Winslow Levee prevents the water from getting to the city. The 1%, 0.5%, and 0.2% ACE floodplains show significant flooding in the left overbank including the City of Winslow. Flooding at the Homolovi I Pueblo begins at approximately the 10% ACE flood.

The Conditional Non-Exceedance Probabilities (CNP) for the baseline condition for the 1% ACE flood is 0.072 for Index Reach 1, meaning that the existing Winslow Levee has a 7.2% assurance or chance of excluding the 1% ACE flood. The CNP for the baseline condition is 0.506 for Index Reach 2.

In order to determine the water surface elevation on the land side of the existing levee, baseline condition FLO-2D hydraulic modeling was completed for the 4%, 2%, 1%, 0.5%, and 0.2% ACE floods. See Plates 16 to 25. For this model, it was assumed the existing levee does fail. The 4% ACE and 2% ACE floods for the FLO-2D baseline condition do not show flooding along the Winslow Levee reach. The 1%, 0.5%, and 0.2% ACE floods show flooding in the City of Winslow caused by failure of the Winslow Levee. The Homolovi I Pueblo is impacted by river flows beginning with approximately the 4% ACE flood event. Sections 6 and 11 discuss the baseline condition hydraulic analyses conducted to determine the flooding along the LCR study reach and at the Homolovi I Pueblo. The floodplains are consistent with the floodplains produced using HEC-RAS. A sediment transport analysis was completed for the baseline condition. See Section 7 of this appendix.

The with-project alternatives (1.1, 3.1, 7, 8, 9, 10, 10.1, 10.2, 10.3, and 10.4) were modeled using HEC-RAS and certain with-project alternatives (3.1 and 10) were modeled using FLO-2D. The 1% ACE flood was modeled for the with-project alternatives to compare with the baseline without-project condition. The alternatives include measures that reduce the flood risk along the LCR. The CNP values for the with-project alternatives are provided in Table 17.

The CNP for Alternative 10.1 for the 1% ACE flood is 0.880 for Index Reach 1, meaning that the existing Winslow Levee has an 88% assurance or chance of excluding the 1% ACE flood. Additional analysis was conducted indicating that 0.3 feet of additional height will increase the assurance to a 90% level for the 1% ACE event, resulting in a levee height that is 3.3 feet above the water surface profile.

With-project alternative floodplains were compared with the baseline condition floodplains to determine changes in water surface elevations, velocities, and flooded areas. Section 11 provides more detail.

1.0 GENERAL DESCRIPTION OF STUDY

1.1 Purpose and Scope

The U.S. Army Corps of Engineers, Los Angeles District is currently conducting the Flood Risk Management Feasibility Phase of the Little Colorado River (LCR) at Winslow Study, a cost shared effort between the U.S. Army Corps of Engineers and the Navajo County Flood Control District.

The purpose of the LCR at Winslow Feasibility Study is to develop and evaluate potential solutions to address flooding issues within and near the City of Winslow.

1.2 Deliverables

This report presents the hydraulic and sedimentation analyses for the present without-project (baseline) condition for the LCR at Winslow area. Specific work included:

- Developing a base model and comparing results against prior existing model information.
- Conducting field and data reconnaissance.
- Plotting 50%, 20%, 10%, 4%, 2%, 1%, 0.5% and 0.2% Annual Chance Exceedance (ACE) floodplain delineations. These floods correspond to the 2-, 5-, 10-, 20-, 50-, 100-, 200-, and 500-year flood frequencies.
- Developing hydraulic input information in support of the economic Hydrologic Engineering Center Flood Damage Reduction Analysis (HEC-FDA) program and risk and uncertainty analysis.
- Generating non-damaging (channel capacity) and/or channel-forming discharge.
- Conducting a quantitative sediment transport analysis based on the baseline condition study discharges.
- Evaluating the lateral channel stability conditions through a qualitative geomorphic analysis.
- Completing hydraulic risk and uncertainty analysis for the baseline condition.
- Revising pertinent hydraulic analyses as necessary based on the review comments.
- Preparing hydraulic documentation in support of all hydraulic efforts.

1.3 Study Area

The LCR originates in the White Mountains, south of Springerville, Arizona. It flows in a north/northwesterly direction in a well-defined canyon until reaching the City of Holbrook, Arizona. From there, it continues west and flows another 30 miles on a broad, open floodplain before it reaches the City of Winslow, Arizona. The river then continues northwest toward Grand Falls, Arizona, where it creates a waterfall around 190 feet in height. The total drainage area of the LCR varies from 11,462 square miles at Holbrook, to 16,192 square miles at Winslow, to 21,068 square miles at Grand Falls, Arizona. Plate 1 shows the location of the LCR Watershed (all plates are located after the text of this hydraulic and sedimentation appendix). The study area is located in the middle of the LCR Watershed, in and near the City of Winslow in west-central Navajo County, Arizona. The study area encompasses the floodplain of the LCR from the vicinity of the Clear Creek confluence downstream (northwest) to the north end of the Winslow Levee. The study area covers the majority of the City of Winslow, including the Ruby Wash

Diversion Levee (RWDL) and the Ruby Wash Levee. The tributaries of Ruby Wash, Clear Creek, Cottonwood Wash, and Jacks Canyon Creek join the LCR Main stem within the study area. See Plate 2 for the Study Area Map.

The City of Winslow is located on the Colorado Plateau in Navajo County, Arizona, at an elevation of 4,880 feet above sea level. Winslow is the largest city in Navajo County, being approximately twice the size of the county seat of Holbrook. Winslow is located on Interstate 40 (I-40) along the western border of Navajo County. Phoenix is located 133 miles to the southwest, Flagstaff is located 55 miles to the west, and Albuquerque is 265 miles to the east.

1.4 Study Background

The Little Colorado River (LCR) at Winslow Feasibility Study is being conducted under authority provided by Section 5 of the Flood Control Act of 1937. This authority amends Section 6 of the Flood Control Act of 1936 to permit the Secretary of the Army, through the Chief of Engineers, to conduct preliminary examinations and surveys for flood control at the Little Colorado River upstream from the boundary of the Navajo Indian Reservation. Further authority is provided under House Committee on Public Works Resolution (Docket 2425) May 17, 1994 which states:

> *"… The Secretary of Army is hereby requested to review reports of the Chief of Engineers on the State of Arizona… in the interest of flood damage reduction, environmental protection and restoration, and related purposes."*

The LCR at Winslow Feasibility Study is one of eight follow-up studies identified in the revised 905(b) Reconnaissance Report for the LCR Watershed Study. The 905 (b) Reconnaissance Report (Reference A), evaluating conditions within the LCR Watershed, was approved by the United States Army Corps of Engineers South Pacific Division on 24 November 1999. The study funds were used to make a recommendation with respect to continued Federal interest in water resource issues including flood control, ecosystem and environmental restoration, storm water retention, water conservation and supply, and recreational needs within the LCR Watershed. A revised 905(b) report for the LCR Watershed was approved 11 August 2008 (Reference B), which found Federal interest and recommended that the study move into the feasibility phase. The City of Winslow has a long history of dealing with flooding along the LCR and its tributaries. There is an immediate need for flood risk management.

1.5 Previous Reports

Many federal and non-federal studies have been conducted pertaining to water and related land resources within the study area. References can be found in Section 14.0

- U.S, Army Corps of Engineers, *Report on Survey, Flood Control, Little Colorado River and its Tributaries Upstream from the Boundary of the Navajo Reservation in Arizona,* Los Angeles District, 1940.
- U.S. Army Corps of Engineers, *Review Report for Flood Control, Winslow, Arizona and Vicinity, Little Colorado River, Arizona and New Mexico,* Los Angeles District, December 1961.

- U.S. Army Corps of Engineers, *Design Memorandum No. 1, General Design for Winslow Flood Control District, Winslow, Arizona, and Vicinity,* March 1969.
- U.S. Army Corps of Engineers, *Flood Plain Information, Little Colorado River, Vicinity of Winslow, Navajo County, Arizona, Los Angeles District*, March 1976.
- U.S. Army Corps of Engineers, *Little Colorado River at Holbrook, Arizona, Review Report for Flood Control and Recreational Development*, Los Angeles District, September 1980.
- George C. Sabol Consulting Engineering, Inc., Scottsdale, Arizona, for Navajo County, Department of Public Works, *Little Colorado River Geomorphology and River Stability Study, Reconnaissance Level Engineering Report*, September 1993.
- U.S. Army Corps of Engineers, *Little Colorado River Watershed, Arizona and New Mexico 905(b) Reconnaissance Report,* Los Angeles District, 1999.
- L.D. & P. J. Garrett, M3 Research, *A Report on Regional Focus Groups to Define Watershed Problems, Opportunities and Concerns in the Little Colorado River Watershed*, 1999.
- U.S. Bureau of Reclamation, *Analysis of Little Colorado River Stability Between Holbrook and Winslow, Arizona, Little Colorado River Sediment Study*, May 2003.
- Navajo County Flood Control District, *Technical Data Notebook with Exhibits, Little Colorado River near Winslow, Floodplain Delineation Study,* November 2005.
- U.S. Army Corps of Engineers, *Supplemental Information to the 1999 Section 905(b) Reconnaissance Report, Little Colorado River Watershed, Arizona & New Mexico, Los Angeles District,* July 2008.
- Navajo County Flood Control District, *Technical Data Notebook with Exhibits, Little Colorado River near Winslow, Floodplain Delineation Study*, July 2009.
- U.S. Army Corps of Engineers, *Little Colorado River at Winslow Feasibility Study Baseline and Future Without-Project Conditions Hydrology Appendix*, Los Angeles District, December 2009.
- U.S. Army Corps of Engineers, *Summary of Winslow Levee and Ruby Wash Diversion Levee, Winslow, Arizona (Navajo County): history, composition, foundation,* Los Angeles District, Geotechnical Branch, March 2010.
- U.S Army Corps of Engineers, *Geotechnical Evaluation of Levee Fragility: Winslow Levee and Ruby Wash Diversion Levee, LCR at Winslow Feasibility, Los Angeles and San Francisco Districts,* Geotechnical Branch, January 2011.
- U.S. Army Corps of Engineers, *Post Charette Report, Little Colorado at Winslow, Arizona*, Los Angeles District, October 2012.

1.6 Homolovi State Park

Significant cultural and historic resource sites are located within the study area, including the Homolovi State Park which features ancestral Hopi Villages. There are multiple pueblos within the state park including the Homolovi I Pueblo which is adjacent to the LCR on the east bank approximately 2 miles downstream (north) of the I-40 Bridges. The Homolovi State Park (See Plate 2 for location) is an important site for the City of Winslow since it is the main local tourism destination.

1.7 Need for USACE Assistance

The Navajo County Flood Control District is attempting to reduce the flood risk along the Winslow Levee and eastern end of the Ruby Wash Diversion Levee due to previous levee failures and de-accreditation. However, the County does not believe that their technical expertise and finances are adequate to complete the project. Navajo County is seeking help from the USACE for financial and technical expertise needed to correct the Winslow Levee system deficiencies.

2.0 DATA COLLECTION

2.1 Topographic Survey

Survey mapping was conducted in 2009 for the Winslow Levee Study area (Reference C) for the Navajo County Flood Control District. Project mapping consists of a 1-foot contour map for the Winslow Levee, a 2-foot contour for the downstream portion of the study area, and the 4-foot contour map for the upstream portion of the study area. Topographic mapping was used to plot overflow delineations. Survey mapping was produced by the Aerial Mapping Company using the North American Vertical Datum (NAVD) 1988 as the vertical datum. The horizontal datum was North American Datum (NAD) 1983 State Plane Arizona East Coordinates. Survey mapping was completed through a Navajo County contract and provided to the USACE for this project.

2.2 As-Built Construction Plans

As-Built construction plans for the I-40 Bridges, the Burlington Northern Santa Fe (BNSF) Railroad Bridge, and the State Route 87 (SR 87) Bridge were reviewed and collected from Navajo County Flood Control District 2009 report (Reference D). The SR 87 Bridge was rebuilt in 2005 and As-Built construction plans were not available. During the site visit from 9-11 August 2011, the bridge dimensions were field measured for use in the hydraulic models.

2.3 Field Investigation

The USACE performed a field visit from 9-11 August 2011 with the intent to observe the Winslow Levee, the LCR Channel, the RWDL, and Ruby Wash, in addition to obtaining additional bridge/underpass data. The study team visited the following locations: Homolovi State Park, BNSF Railroad Bridge, SR 87 Bridge, I-40 Bridges, Winslow Levee, and Ruby Wash Levee. The study team visited Homolovi State Park to observe how the river impacts the Homolovi I Pueblo, which is a major local tourist destination near Winslow. During the visit of the Winslow Levee, the team specifically visited the area where the LCR impinges on the levee. The following were measured during the field investigations: LCR bridge crossings, underpasses along I-40, culverts beneath SR 87 and the BNSF Railroad.

The levee has experienced overtopping (1993) and piping (2003) along approximately a 10,000 foot stretch between the two impingement points. This stretch of the Winslow Levee has been reinforced with riprap on both embankments. The two impingement locations have been reinforced as well, but they still need to be monitored due to the river’s proximity along the levee. The saltcedar and vegetation in the floodplain is denser than previously assumed near the bridges and along both banks near the two impingement locations. See Attachment 1 of this report for a copy of the site visit report which includes photos.

2.4 LCR Winslow Charette

The Project Delivery Team (PDT) met from 29-31 May 2012 for a three day plan formulation charette workshop that was held in Winslow, Arizona. The primary purpose of the charette was to use this collaborative process to expedite plan formulation for the preliminary array of alternatives. The intent of the charette was to formulate alternatives and identify study objectives as well as address problems, opportunities, and constraints. Participants in the charette workshop included representatives from the USACE, Navajo County Flood Control District, City of

Winslow, Arizona Game and Fish Department, Arizona Department of Environmental Quality, Arizona Department of Water Resources, Hopi Tribe, Navajo Nation, Pueblo of Zuni, Arizona State Parks, United States Geological Survey (USGS), Arizona State Museum, and BNSF Railroad.

2.5 Sediment Samples

The United States Bureau of Reclamation (USBR) conducted a sediment study for the LCR between Holbrook and Winslow in 2003 (Reference E). Bed material from the Little Colorado River and its tributaries was collected by the USBR and Navajo County personnel and analyzed at a Navajo County soils laboratory for grain size distribution. Appendix C of the USBR report contains the inventory of the bed material samples taken as well as the sieve analysis results. In total, 57 surface bed material samples were collected over more than 50 miles of the LCR. Sediment data collected by the USBR near the Winslow area was used for the current study. The data was analyzed and confirmed by the USACE Geotechnical Study Engineer.

3.0 EXISTING CONDITION OF LEVEES

3.1 Winslow Levee

The Winslow Levee was built, rebuilt, and is maintained and owned by Navajo County, Arizona. Plate 3 shows the existing Winslow Levee alignment. In 1979, Navajo County requested assistance from the Arizona Department of Water Resources (ADWR) to build the Winslow Levee. After completing the necessary engineering and securing needed funding and right-of-way, the 7.2-mile Winslow Levee was constructed along the west side of the LCR between 1986 and 1989. The levee design included bank protection and cutoff walls and it was designed to contain the estimated 1% ACE flood of 65,000 cubic feet per second (cfs) back in the 1980s. Hydrologic analysis completed by the USACE in 2010 shows that the 1% ACE discharge near Winslow is about 69,200 cfs. The *Summary of Winslow Levee and Ruby Wash Diversion Levee, Winslow, Arizona (Navajo County): history, composition, foundation* (Reference F) provides extensive discussion regarding the Winslow Levee history and repairs.

3.1.1 Flood History

Based on the Floodplain Information Study (FIS) published by the USACE in March 1976 (Reference G), significant floods on the LCR at Winslow occurred in 1923, 1957, 1968, and 1969. Flood records also show a peak discharge of 57,000 cfs occurred during the flood of December 1978, which overtopped and circumvented an existing 4-mile long levee system installed by the Navajo County Flood Control District (Reference H).

Only four years after construction, on January 8, 1993, the levee was overtopped by a flood having an estimated peak discharge between 70,000 cfs and 77,000 cfs (Reference D). See Figure 1 for the LCR Discharge Frequency Curve provided by the USACE 2009 Hydrology Appendix (Reference I). As a result of the levee overtopping, a 400 foot section of levee was washed out, while a 3,000 foot section of levee was damaged. Properties were flooded in Ames Acres, Bushman Acres, and other areas behind the levee. In total, 204 parcels were inundated and 140 structures were damaged. A lawsuit resulted, which required $1,400,000 in Navajo County funds to settle. Temporary repairs to the levee were completed immediately following the flooding. Permanent repairs were completed in 1994 using Federal Emergency Management Agency (FEMA), State, and County funds. The repairs included adding riprap to both sides of the levee along a 3,000 foot reach near the failure. See Plate 4 for flooding location.

On December 31, 2003, the levee experienced a piping failure at well below a 1% ACE flood (around 4% ACE to 2% ACE flood). The water depth was approximately 16 feet compared to the anticipated 1% ACE flood water depth of 25 feet at the piping location. An alert citizen reported the impending levee failure and Navajo County responded immediately, and levee failure was avoided by depositing material on the river side of the levee. Permanent repairs were completed in 2005 as riprap was extended along both sides of the levee. The riverside of the levee has protection from design station 140+00 to 400+00, and the landside of the levee has protection from design station 176+00 to 231+60. The piping was induced by the sandy subsoil beneath the levee, while the bentonite core was found intact. See Plate 4 for flooding location from the 1993 flood.

3.1.2 Floodplain Studies

Four tributaries join the LCR upstream from the City of Winslow, including Ruby Wash, Clear Creek, Chevelon Creek, Jacks Canyon, and Cottonwood Creek. The tributaries had a substantial contributory effect to the December 1978 flood according to ADWR. Hydraulic models completed for the Navajo County Flood Control District in 2009 (Reference D) indicate that the levee, in its current condition, will overtop in approximately the same location that it did in January of 1993 at a discharge of 55,000 cfs. This is approximately the 2% ACE flood. The floodplain study concluded that the Winslow Levee does not have the capacity to contain the 1% ACE flood and does not meet FEMA standards for 1% ACE flood protection.

A study completed by the USBR, *Analysis of Little Colorado River Stability between Holbrook and Winslow, Arizona* (Reference E), determined that there has been no significant sediment aggradation since the 1980s. However, there are other factors that account for the difference between the current results and the results of the previous floodplain delineation study. Just downstream of where the levee breached in 1993, there is a topographic feature that restricts the flow and creates a meander loop that forces the river against the levee. This restriction, which was identified by ADWR as the cause of the levee failure in 1993, does increase the floodwater elevations in this reach. Typically, meander loops get cut off by big floods. However, the topographic feature creating this loop appears to be fairly resilient and has been stabilized somewhat by the dense vegetation covering it. This feature survived the 1993 flood, and remains in existence today. The previous floodplain study completed in 1976 (Reference G) modeled the meander loop as a transient feature (something that would get washed out in a big flood). This resulted in a river model with more conveyance capacity than is really available. The Navajo County floodplain study in 2009 (Reference D) used higher Manning's roughness coefficients than were used by the previous study (Reference G). These coefficients can have a significant impact on calculated water surface elevation (WSEs).

3.1.3 Conditional Non-Exceedance Probability and Levee Freeboard

Engineering Circular 1110-2-6067 (Reference J) was used to determine the required levee height above the water surface (freeboard). The chance of non-exceedance of the levee elevation, the uncertainty in the discharge-probability function and the stage-discharge function are combined to get the uncertainty in the stage-probability function. Hydrologic Engineering Center's Flood Damage Analysis program was used to compute the combined uncertainty as well as the assurance or conditional non-exceedance probability (CNP) of the levee excluding the 1% chance exceedance flood from the leveed area. To meet the National Flood Insurance Program (NFIP) levee system evaluation requirements, a levee must have at least 90% assurance of excluding the 1% ACE flood for all reaches of the system. For levees, if the top of levee elevation is less than the FEMA required freeboard above the 1% ACE flood stage, then the levee can only be in accordance with NFIP levee system evaluation requirements if the assurance (CNP) is 95% or greater. The top of levee elevation shall not be less than two feet above the 1% ACE flood elevation, even if assurance is 95% or greater.

FEMA's standards for accrediting levees for 1% ACE flood protection require that they have a minimum of 3 feet of freeboard. In addition to this, when the Winslow Levee was designed, it was determined that another 2 to 3 feet of freeboard would be needed to provide storage for sediment that would build up within the channel over the life of the levee. Therefore, for most of the levee, the design freeboard was 5 to 6 feet. However, the Navajo County Flood Control District determined that the levee needs to be raised substantially along much of its length after surveyed top-of-levee elevations were compared with the calculated 1% ACE floodwater elevations according to the 2009 report (Reference D).

3.1.4 Levee Improvements

The levee improvements that are needed to provide 1% ACE flood protection cannot be accomplished by simply adding material to the top of the levee. The improvements involve reconstructing the levee. Additional details are provided in the Design Appendix and additional design studies and construction plans will be needed before this work can begin.

Impact to Digital Flood Insurance Rate Maps: The Map Modernization deployed by FEMA deaccredited the Winslow Levee in September 2008 and put 2,700 new parcels in the floodplain.

3.2 Ruby Wash Diversion Levee (RWDL)

In addition to the Winslow Levee, several other structures contribute to the current level of flood protection for the City of Winslow, including the RWDL and the Ruby Wash Levee. The USACE designed and constructed the RWDL. This levee is a rock and earth structure extending 5.3 miles from the high ground near the southwest corner of the Winslow airport to the Little Colorado River south of the BNSF Railroad Bridge east of Winslow. The construction of this levee was completed in 1970. Flows in Ruby Wash and in other streams crossing the alignment of the levee are diverted east to the Little Colorado River, eliminating flood hazards along Ruby Wash. The Ruby Wash Diversion Levee protects the Winslow Airport and approximately 500 residents. See Plate 3 for the location of the RWDL and the RWDL training dike that was constructed at the downstream end of the Ruby Wash Diversion along the right bank near the confluence with the Little Colorado River.

The Ruby Wash drainage area is approximately 26 square miles and consists of low desert valleys traversed by shallow ravines with elevations ranging from about 4,880 feet to 5,250 feet above mean sea level. The design discharge for the Standard Project Flood is 8,500 to 23,000 cfs.

The Ruby Wash Diversion Levee (RWDL) is at risk of failure due to the changed conditions on the LCR main stem described for the Winslow Levee above. While the RWDL continues to serve its intended purpose of diverting damaging Ruby Wash flows away from Winslow, this levee was not designed or intended to address flooding along the LCR main stem. The easternmost portion of the RWDL is subject to overtopping from LCR main stem flows. The RWDL could also fail before overtopping during a flood as frequent as the 4 percent ACE (25-year) event. The eastern end of the RWDL (where it abuts the Winslow Levee) has been identified as the levee segment most susceptible to failure. A levee failure at this location could cause damage to the City of Winslow and other areas behind the levee. The improvements needed at RWDL are likely to involve reconstruction of the levee.

3.3 Ruby Wash Levee

The Ruby Wash Levee was constructed by the Arizona Department of Transportation (ADOT) in 1980 as part of the Interstate 40 at Winslow Project (Project I-40-4(81)). The Ruby Wash Channel extends from Third Street to I-40. Due to the flat terrain along the channel alignment, the channel was constructed using a small amount of excavation below the existing ground surface. The majority of the channel construction was accomplished by creating embankments of compacted earth above the natural ground elevation to form the channel banks, which are referred to as levees. See Plate 3 for the location of the Ruby Wash Levee.

In the late 1990's, Navajo County made substantial engineered improvements to the Ruby Wash Levee resulting in the levee providing flood protection for a portion of downtown Winslow. The levee met 44CFR 65.10 requirements prior to the FEMA Map Modernization program. The Ruby Wash Levee is not in the USACE Rehabilitation Program.

4.0 HYDROLOGY

4.1 Description of Drainage Area

The hydrologic analysis encompasses the watershed of the LCR and major tributaries, including Chevelon Canyon, Ruby Wash, Clear Creek, Cottonwood Wash, Salt Creek, and Jacks Canyon. The LCR originates in the White Mountains, south of Springerville, Arizona. It flows in a north/northwesterly direction in a well-defined canyon until reaching the City of Holbrook, Arizona. From there, it continues westerly and flows another 30 miles on a broad, open floodplain before it reaches the City of Winslow, Arizona. The river continues northwesterly towards Grand Falls, Arizona, where it creates a waterfall around 190 feet in height at the confluence with the Colorado River. The total drainage area of the LCR varies from 11,462 square miles at Holbrook, to 16,192 square miles at Winslow, to 21,068 square miles at Grand Falls, Arizona.

The overall basin characteristics are summarized below:

- The basin is a portion of Colorado Plateau characterized by various rock formations and broad valleys with extensive flat, mesa-like highlands.
- Vegetation cover ranges from barren desert to mountain forest, including juniper, sagebrush, and grass. LCR and its tributaries support annual grass and shrubs.
- Elevation above mean sea level ranges from 11,500 feet at the origin of the LCR to 4,800 feet at Winslow.
- The LCR basin is generally cool in the winter and warm in the summer. Temperatures range from 110° F in the lower part of the basin, in summer, to around -35° F in the upper part of the basin, in winter.
- The primary rainy season is the summer “monsoon,” which occurs from July to September.
- A strong rainfall period is observed during the winter months. Typically, late spring and June are dry throughout the basin.
- Mean-annual precipitation ranges from around 7 inches near Winslow to around 40 inches in the upper portion of the basin.
- Average stream flows in the LCR and its tributaries are minimal, and sometimes the stream flows reduce to zero.
- Detailed hydrologic analysis for the study area including flood frequency analysis, volume flow frequency analysis, and balanced hydrographs are presented in the Hydrology Appendix (Reference I).

4.2 Hydrologic Data Input for HEC-RAS Model

Hydrologic data from the *Little Colorado River at Winslow Feasibility Study Baseline and Future Without-Project Conditions Hydrology Appendix* (Reference I) was used in the hydraulic analysis including discharge frequency values at the designated concentration points and the balanced hydrographs. Table 1 shows the discharge frequency values at the designated concentration points for the steady state HEC-RAS simulations. The table shows the river, reach, concentration points, and discharge frequency values.

4.3 Balanced Hydrograph for the FLO-2D Model and Sediment Transport Model

For the floodplain analysis, specifically FLO-2D model simulations, inflow hydrographs at the upstream boundary of the channel are required. In order to preserve the peak flow and volumes of the flooding events, the balanced hydrographs were calculated for the simulation. Table 2 presents the peak and volume discharge frequency values for the LCR at Winslow. The calculated 50%, 20%, 10%, and 4% ACE balanced hydrographs are plotted graphically and shown in Figure 2, while the calculated 2%, 1%, 0.5%, and 0.2% ACE balanced hydrographs are shown in Figure 3.

5.0 HYDRAULIC ANALYSIS USING HEC-RAS – BASELINE CONDITIONS

This section describes the channel hydraulic analysis for the present without-project baseline condition. A one dimensional steady flow model was developed using HEC-RAS version 4.2 beta. Hydrologic Engineering Center Geospatial River Analysis System (HEC-GeoRAS) was applied to assist the HEC-RAS model development.

Water surface profiles were developed for the 50%, 20%, 10%, 4%, 2%, 1%, 0.5% and 0.2% ACE floods. These correspond to 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year flood frequency events, respectively. The 1% ACE flood has a 1 in 100 chance of being equaled or exceeded in any 1 year, and it has an average recurrence interval of 100 years, it often is referred to as the "100-year flood". Scientists and engineers frequently use statistical probability (chance) to put a context to floods and their occurrence. If the probability of a particular flood magnitude being equaled or exceeded is known, then risk can be assessed. To determine these probabilities, all the annual peak stream flow values measured at a stream gage are examined. A stream gage is a location on a river where the height of the water and the quantity of flow (stream flow) are recorded.

5.1 Hydraulic Model Overview

Analysis for the baseline condition was initially broken into two models (Model 1 and Model 2). Model 1 includes the 50%, 20%, 10%, 4%, and 2% ACE floods. The Winslow Levee was included in the HEC-RAS model. Model 2 includes the 1%, 0.5%, and 0.2% ACE floods with the Winslow Levee removed from the model. The analysis was separated into two models for floodplain development. Model 2 was developed with the Winslow Levee removed from the model based on the levee history detailed previously in Section 3.1 as well as the 2009 Navajo County analysis performed by Delph Engineering, which concluded that the Winslow Levee does not have the capacity to contain the 1% ACE flood and does not meet FEMA standards for 1% ACE flood protection. A third hydraulic model (Model 3) was created that included the Winslow Levee for the 1%, 0.5%, and 0.2% ACE floods to capture the water surface elevations before levee overtopping. Further description of the three models is provided in the paragraphs below.

Only four years after construction of the Winslow Levee, on January 8, 1993, the levee was overtopped by a flood having an estimated peak discharge between 70,000 cfs and 77,000 cfs (Reference D). As a result, a 400 foot section of levee was washed out, while a 3,000 foot section of levee was damaged. According to the Arizona State Museum's *Flooding and Threats to Archaeological Sites* report (Reference K), the 1993 flood inundated the South plaza of the Homolovi I Pueblo, which is an archeological site on the riverside of the Winslow Levee that is in the 1% ACE floodplain. The floodplain from Figure 8 in the Museum's report matches the floodplain from Model 3.

For the baseline condition 1% ACE flood hydraulic model (Model 2), the Winslow Levee was removed from the geometry file in HEC-RAS due to the model showing levee overtopping upstream from the BNSF Railroad Bridge. By assuming levee failure and removing the Winslow Levee in HEC-RAS, flows were allowed to convey across the entire floodplain which resulted in flood flows inundating the City of Winslow. Model 2 shows the state of the flooding after the levee has already failed, and it provides an estimation of flood extents for the 1% ACE floodplain on the land (west) side of the current levee. However, water surface elevations on the river side of the levee are not accurately represented in this steady state model. During the 1% ACE flood through the LCR Winslow reach, the water surface elevations increase with time during the flood according to the flow hydrograph. In the meantime, the water level near the Homolovi I Pueblo is increasing too until the levee starts to fail and flows break out into the City of Winslow area. This is a dynamic process or an unsteady state condition. Once the levee fails in this scenario, the water level behind the levee starts to decrease (including near the Homolovi I Pueblo).

The baseline condition hydraulic model was simulated using steady state modeling in order to provide data for economic evaluation. For a steady state simulation of the 1% ACE flood, the water level near the Homolovi I Pueblo for the baseline condition is the water level after the levee failure when the flows are allowed to convey across the entire floodplain. The Homolovi I Pueblo is located along the right bank of the LCR just upstream from hydraulic river station 390+00. Model 2 does not accurately record the maximum water surface that occurs as the flows increase to the 1% ACE peak flow before the levee failure as it models the post breakout condition in a steady state hydraulic analysis.

Consequently, a third hydraulic model (Model 3) was created that included the Winslow Levee for the 1%, 0.5%, and 0.2% ACE floods. Flows ranging from the 4% ACE flood (38,310 cfs) to the 1% ACE flood (69,200 cfs) were used to estimate the water surface elevations on the river side of the levee to get a stage-discharge relationship. This model assumed levee failure would not occur. See Figure 4 for the stage-discharge curve. The stage-discharge curve shows the water surface elevations as the discharges increase from the 4% ACE flood to the 1% ACE flood. This was done to show how the water surface elevations would increase near the Homolovi I Pueblo as the flows increase to the 1% ACE peak flow (69,200 cfs) assuming no levee failure.

The following sections introduce the HEC-GeoRAS program, describe the model development, and present the model simulation results.

5.2 Introduction to Modeling Procedures

HEC-GeoRAS is an ArcView Geographic Information System (GIS) extension that provides the user with a set of procedures, tools, and utilities for the preparation of GIS data for import into HEC-RAS. HEC-GeoRAS version 10.0 was used within ArcMap 10.0 to develop spatial data obtained from the 2009 topographic survey. Using HEC-GeoRAS, a RAS export file was generated that contained river, reach, and station identifiers; cross-sectional cut lines; cross-sectional surface lines; cross-sectional bank stations; downstream reach lengths for the left over bank, main channel, and right over bank; and cross-sectional roughness coefficients.

After importing the GIS geometry data into HEC-RAS, the geometric data set and flow data were completed before performing hydraulic computations. Water surface and velocity results from HEC-RAS simulations were exported back to GIS to develop flood inundation maps for the eight flood frequencies using HEC-GeoRAS. Detailed mapping is available in electronic format.

5.3 HEC-RAS Model Development

HEC-RAS version 4.2 beta is a one dimensional hydraulic model that can model a full network of channels. The HEC-RAS model begins just downstream from the Clear Creek Confluence with the LCR, where the river flows northwest toward the City of Winslow. The LCR Winslow study reach begins approximately 10,000 feet upstream (southeast) from the BNSF Railroad Bridge and ends approximately 50,000 feet downstream (north) from the I-40 Bridge.

HEC-GeoRAS was used to assist the HEC-RAS model development as described in Section 5.1. HEC-RAS model development included the processing of cross-sections and bridges, defining Manning n-values and flow regime, evaluating ineffective flow areas, and setting boundary conditions.

The hydraulic model consisted of approximately 12 miles of the LCR. After a review of previous studies done by the Bureau of Reclamation (Reference E) and Delph Engineering (Reference D), the study team visited the site on 9-11 August 2011. The primary purpose of the visit was to observe the Winslow Levee, the LCR floodplain, and the BNSF Railroad and highway bridges/underpasses. Furthermore the site visit assisted in the estimation of Manning's roughness coefficients (n-values) and general site conditions. The following sections provide detail on the HEC-RAS hydraulic modeling.

5.3.1 Cross-Sections

Digital terrain data (TIN) and aerial maps were used to generate cross-sections and approximate stream centerlines for the LCR. The survey data was prepared for Navajo County in 2009. The centerlines for the LCR were determined using ESRI aerial imagery (January 2013). Cross-sections were placed approximately every 500 feet along the LCR as well as upstream and downstream of bridges. After cross-sections were processed by HEC-GeoRAS from the TIN, they were then exported to HEC-RAS with station and elevation data for each cross-section.

The hydraulic river stations correspond to the cumulative stream length measured from the downstream end of the study boundary. Hydraulic river station 5+00 is the downstream boundary of the LCR, which is approximately 10 miles from the I-40 bridges. Hydraulic river station 630+00 is the upstream station, which is approximately 2 miles upstream from the LCR confluence with Ruby Wash. Plate 5 shows the cross-sections for the HEC-RAS model

5.3.2 Roughness Coefficients (Manning's n-values)

The roughness coefficients (Manning's n-values) for the main channel, left overbank, and right overbank were estimated reach-by-reach based on the topographic mapping, aerial photos, as-built drawings, and field investigations. The left and right overbanks have horizontally varied n-values, while the main channel has a constant n-value throughout the model. The main channel n-values were determined using the recommended values from the ADWR "*Design Manual for Engineering Analysis of Fluvial Systems*" (Reference L) and *Open Channel Hydraulics*, Chow (Reference M). The n-value used for the urban areas in the City of Winslow is based on Hejl's *A method for adjusting values of Manning's roughness coefficients for flooded urban areas (Reference N).* The selected n-values were kept constant between the eight flood frequencies. Table 3 and Plate 6 show the Manning's n-values for the LCR at Winslow Study reach. The

Manning's n-values used for this study were compared to those used in the USBR 2003 study which had overbank n-values vary between 0.077 to 0.15 and a main channel n-value of 0.025.

5.3.3 Bridges

The bridge data was based on the as-built drawings and data provided by Navajo County. See Table 4 for a list of bridges that cross the LCR near Winslow and a summary of properties such as number of piers, bridge span length, pier shape, pier width, and bridge width. As-built drawings (Reference D) were used to determine pier spacing for the four bridges, and field measurements verified the spacing. The new SR 87 Bridge was completed in 2005 for which no as-built information was provided. Field measurements were completed to verify pier spacing and bridge deck thickness. See Plate 7 for bridge locations.

Pier debris was added to the Interstate 40 Bridges in the HEC-RAS model that was completed as the piers are 2 feet wide. Pier debris was not added to the BNSF Railroad Bridge or the SR 87 Bridge as the piers for those bridges are seven and six feet wide, respectively. This assessment was based on Los Angeles District Hydrology and Hydraulics Policy Memorandum No. 4, *Debris Loading on Bridges and Culverts.*

5.3.4 Flow Regime

The hydraulic models were run using a subcritical flow regime due to the Froude number (dependent on flow velocity, acceleration due to gravity, and depth of flow) in the channel being less than 1.0 along the LCR at Winslow reach. Water surface profile computations begin at a cross-section with known or assumed starting conditions and proceed upstream for subcritical flow.

5.3.5 Boundary Condition

Boundary conditions are necessary to establish the starting water surface at the ends of the river system (upstream and downstream). In a subcritical flow regime, an upstream discharge boundary condition and a downstream water surface elevation are needed.

The flow rates for the upstream cross-section 630+00 were provided by the LCR near Winslow gage from Table 5 of the USACE Hydrology Appendix (Reference I). The discharge frequency values for the HEC-RAS model were summarized in Table 1 of this report. The downstream boundary condition was normal depth. An energy slope of 0.001 was used as the downstream boundary condition based on the average channel slope from hydraulic river station 10+00 to 0+00.

5.3.6 Ineffective Flow Area Boundaries

Ineffective flow areas were used in the model to account for non-conveying flow areas. These locations were determined to be areas where the flow has zero velocity. Ineffective areas were modeled due to backwater behind bridges and small tributaries and in locations that experienced sudden contraction or expansion of flow as well as in locations with flow restrictions created by natural dune landforms. Such areas were determined using aerial photography, the contours, and the TIN.

5.3.7 Calibration

The HEC-RAS model was calibrated by varying Manning's n-values and by using the 2009 HEC-RAS model provided by Navajo County Flood Control District (Reference D). Observed water surface elevation data was not available. However, Exhibit 8 from the 2009 report *Flooding Threats to Archaeological Sites* (Reference K) shows the approximate floodplain in the Homolovi I Pueblo area for the 1993 flood (Hydraulic river station 390+00). The floodplain results for the baseline condition analysis were compared (but not calibrated) to Exhibit 8. The 1993 flood had an estimated peak discharge between 70,000 cfs and 77,000 cfs (Reference D) (1% ACE floods has a peak discharge 69,200 cfs).

5.3.8 Model Assumptions

The BNSF railroad embankment upstream from the BNSF Railroad Bridge (along the right bank) was modeled as a levee within HEC-RAS from hydraulic river station 630+00 to 570+00 (along the right bank). Furthermore, I-40 was modeled as a levee for similar reasons to better represent the conveyance area near the City of Winslow downstream from the I-40 Bridge from hydraulic river station 495+00 to the beginning of the Winslow Levee at hydraulic river station 470+00.

5.4 Baseline Hydraulic Model Results and Floodplain Analysis

Flow breakout analysis for the LCR was conducted using HEC-RAS and HEC-GeoRAS. Water surface profiles were produced and exported to GIS for floodplain mapping for the 50%, 20%, 10%, 4%, 2%, 1%, 0.5%, and 0.2% ACE floods. Floodplains for the eight frequencies are displayed on Plates 8 to 15, respectively. On the river side of the levee, the floodplain was modeled using Model 3 (no levee failure); Model 2 (no levee) was used to get flood depths on the land side of the levee (City of Winslow). The two floodplains were combined using GIS and are discussed below.

5.4.1 Floodplain Analysis

The 1%, 0.5%, and 0.2% ACE floodplains (See Plates 13, 14, and 15) show significant flooding in the left overbank including the City of Winslow. The flooding near the I-40 bridges is caused by backwater on the bridges at the BNSF Railroad and the I-40 bridges. The flooding in the right overbanks does not threaten any structures although it does encroach on the Homolovi I Pueblo near hydraulic river station 390+00. The 1%, 0.5%, and 0.2% ACE floods overtop the Ruby Wash Diversion Levee, resulting in flooding in the left overbank including the City of Winslow. When combined with the overtopping of the Winslow Levee between the BNSF Railroad Bridge and the SR 87 Bridge, the floodplain shows extensive flooding in the City of Winslow.

The 2% ACE flood shows the Winslow Levee/Ruby Wash Diversion Levee being overtopped due to backwater caused by the BNSF Railroad Bridge. The breakout occurs at hydraulic river station 535+00. A sensitivity analysis was completed to determine at which point the levee becomes overtopped. At approximately 44,000 cfs, the Winslow Levee begins to overtop (a 2% ACE flood is approximately 52,020 cfs). The existing levee is deficient in height along in the section of the levee upstream from the BNSF Railroad Bridge (hydraulic river station 529+83 to 542+50). Plate 12 shows the flooding is contained by the BNSF Railroad and flows do not reach

Winslow. The amount of flow through the BNSF Railroad underpasses is negligible considering the duration of the overflow and the volume that is stored south of the railroad embankment.

The 50%, 20%, 10%, 4%, and 2% ACE floodplains do not show flooding in the City of Winslow as the levee prevents the water from getting to the city. The flooding in the right overbanks does not threaten any structures for the 50% ACE and 20% ACE floods. However, flooding at the Homolovi I Pueblo begins at approximately the 10% ACE flood, causing increased flooding at this location for the 10% ACE to 0.5% ACE floods.

5.4.2 Freeboard Condition

FEMA's standards for accrediting levees for 1% ACE flood protection require that levees have a minimum of 3 feet of freeboard. The freeboard for the baseline condition is below 3 feet for the 1% ACE flood along the majority of the Winslow Levee (Section 3.1.3). Upstream from the BNSF Railroad Bridge, the Winslow Levee is overtopped (station 542+50 to 529+39) during the 1% ACE flood event. The majority of the levee downstream from the BNSF Railroad Bridge does not currently meet the 3 feet freeboard requirement for FEMA accreditation.

6.0 HYDRAULIC ANALYSIS USING TWO DIMENSIONAL FLOW

The Winslow Levee study area is in a flat alluvial fan area, and the flooding is a dynamic process. In an overflow analysis where flooding is not limited by topographic changes (flat) in term of flow direction, a two-dimensional model is more appropriate to estimate flood limitation. The unsteady state simulation using the inflow hydrograph with flow volume conservation can produce a mathematical simulation closer to the real flooding scenario. Because of these reasons, FLO-2D was applied to the study area due to the two-dimensional application, transient simulation, and volume conservation. FLO-2D results complement the HEC-RAS model results discussed in Section 5.0 and also provide a better understanding of the Winslow Levee system. This section presents the introduction of the FLO-2D program, model development, and simulation results. FLO-2D is a Corps of Engineers approved model for surface water hydraulic applications.

6.1 Introduction to FLO-2D

FLO-2D is a two-dimensional flood routing model that was used to perform additional hydraulic analysis, delineate floodplains, and determine flow depths. FLO-2D is a volume conservation flood routing model that distributes a flood hydrograph over a system of square grid elements. It was used to create and process the surface water components. FLO-2D is a flood routing model that simulates channel flow and unconfined overland flow. FLO-2D uses the continuity and momentum equations, and numerically routes a flood hydrograph while predicting the area of inundation and simulating flood wave attenuation.

Channel flow is one-dimensional with the channel geometry represented either by natural, rectangular, or trapezoidal cross-sections. Channel overbank flow is computed when the channel capacity is exceeded. An interface routine calculates the channel to floodplain flow exchange including return flow to the channel. Once the flow overtops the channel, it will disperse to other overland grid elements based on topography, roughness, and obstructions.

FLO-2D simulates unconfined overland flow using topographic data files that have been developed from a digital terrain model or digitalized base map. The FLO-2D software package includes a grid developer system (GDS) that will overlay a square grid system on a set of digital terrain (DTM) points that were derived from the 2009 survey data (Reference C). The GDS will filter DTM points, interpolate the DTM data, and assign elevations to grid elements (Reference O).

The governing fluid equations of the model are the continuity equation and momentum equation in two-dimensional form. The momentum equation used in the model is in a form known as the dynamic wave momentum equation. FLO-2D models channel flow using the one-dimensional dynamic wave approximation to the momentum equation. The channel-floodplain exchange is based on the potential water surface elevation difference between the channel and the floodplain grid element containing either channel bank. The computed velocity of either the outflow from the channel or the return flow to the channel is computed using the diffusive wave equation, which neglects the last three acceleration terms in the momentum equation. Overbank flow modeling is solved using the 2-dimensional dynamic wave approximation to the momentum equation.

6.2 FLO-2D Model Development

6.2.1 Model Overview

FLO-2D was used in this study to model overbank flows, which are comprised of flows that travel out of stream channels and across the topography of the floodplain. FLO-2D has the capability of modeling both one-dimensional channel flow and two-dimensional overbank flow.

6.2.2 Procedure and Process

The first task in developing the FLO-2D model was assembling the topographic data for the Winslow Levee Study. The FLO-2D grids in overbank areas were constructed from the 2009 survey data provided by Navajo County (Reference C). Using the FLO-2D pre-processing program GDS to process the DTM, a 300-ft by 300-ft grid element system was developed for the study area. Since the FLO-2D model was developed to simulate the floodplain for planning purpose, and due to the computational time consideration, the 300-ft by 300-ft grid is appropriate for this study. The channel data file was created directly from the HEC-RAS model using the GDS pre-processing program. The levee data input file was created using the geo-referenced shape file and FLO-2D levee data input guideline.

Water surface output from the FLO-2D model was exported to a GIS environment. Post-processing of the output in conjunction with the topographic data and aerial photos was performed to generate and define floodplains.

6.2.3 Boundary Conditions

Boundary conditions for the model include inflow and outflow boundary nodes, tail water conditions, and channel inflow hydrographs. Inflow boundary nodes are identified in the input file and inflow hydrographs are provided from the hydrologic analysis. Tail water conditions for the outflow nodes are based on normal depth, with the slope computed from adjacent node elevations. The inflow boundary node for the Winslow Levee Study area was set upstream of the SR 87 Bridge. The balanced hydrographs (shown in Figures 2 and 3) were entered into the model at the inflow boundary node. The outflow boundary condition was set at the downstream model boundary.

6.2.4 Assumptions and Limitations

Several basic assumptions and limitations must be considered with the FLO-2D model. Two-dimensional flow simulation in FLO-2D is limited to the eight directions of the compass. The model routes channel and overland flow using the full dynamic wave or the diffusive wave approximation to the momentum equation. The simulations performed presented a fixed bed analysis. Bridges and overland flow on streets were not included in the model.

Since the input hydrograph has 84 hours (3.5 days) duration, the developed model was run to simulate the same time period. During the simulation run, the FLO-2D model uses a time step value dependent on a Courant Number of 1. Other measures implemented to insure stability include limiting the floodplain and channel depth change per time step to 20% and 25%, respectively, and limiting the time step to a maximum of 30 sec. The minimum flow depth for flood routing is 0.1 ft.

6.3 Floodplain Analysis Approach

The Winslow Levee is about 7.2 miles long, and was designed to protect against a 65,000 cfs discharge in the LCR. Based on the current hydrologic analysis, the 1% ACE peak discharge is about 69,200 cfs. The Winslow Levee design discharge is below 1% ACE peak discharge. Previous levee issues include an overtopping failure which occurred between 2% to 1% ACE flood frequency events and a piping failure which occurred between 4% ACE to 1% ACE flood frequency events (see Section 3.1.1). For the FLO-2D simulation runs, five different scenarios of levee simulations were assumed. The simulation results were used to check with HEC-RAS results for the compatibility of the two models. The five FLO-2D modeling scenarios are as follows:

- The first scenario assumes that LCR flows are contained by the levee.
- The second scenario assumes that the levee fails totally.
- The third scenario assumes that the levee fails at four different locations due to impingement and piping failures.
- The fourth scenario assumes that the levee fails due to impingement near the SR 87 Bridge area.
- The fifth scenario assumes that the levee fails at three locations downstream of the Bushman Acres community.

The second, third, fourth, and fifth scenarios were used to complement the HEC-RAS simulations. Under the steady state condition, HEC-RAS is difficult to simulate the dynamic levee failure at different locations. The assumption of these scenarios were based on the historical failures and known information (see Section 3.1.1).

6.3.1 Scenario 1: Levee Remains

The first scenario assumes that LCR flows are contained by the levee and that the levee remains intact. The levee was assumed functional for the 50%, 20%, 10%, 4%, and 2% flood discharge conditions. The simulation results were compared with the HEC-RAS results (Model 1) as presented in Chapter 5. The results from these two models are almost identical. Plate 16 shows the simulated floodplain for the 4% ACE flood. Plate 17 shows the simulated floodplain for the 2% ACE flood. The corresponding HEC-RAS simulation results are shown in Plates 11 and 12, respectively. The comparisons demonstrate that the simulation results from the two models are compatible and in agreement. Since the scenario is for model comparison purpose, the 50%, 20%, and 10% ACE floodplain results are not presented in this report.

6.3.2 Scenario 2: Total Levee Failure (No Levee)

The second scenario assumes that the levee fails totally and does not include a levee in the model. The levee was assumed totally failed for the 1%, 0.5%, and 0.2% ACE floods. This scenario uses the same assumption as HEC-RAS Model 2 described in Section 5. Plate 18 shows the simulated floodplain for the 1% ACE flood. As shown in the figure, water flows between SR 87 and I-40 into the City of Winslow and inundates half of the city area. North of I-40, floods inundate large areas in the overbanks, including properties in Ames Acres, Bushman Acres, and other areas behind the levee. However, near the lower end of the levee and the west river bank area, some areas are not inundated based on this simulation.

Plate 19 shows the simulated floodplain for the 0.5% ACE flood. Plate 20 shows the simulated floodplain for the 0.2% ACE flood. Comparisons between Plate 18, Plate 19, and Plate 20 show that the flooding areas are almost the same for the three flood frequencies. The differences are the flow depth. A review of the balanced hydrographs of Figure 3 shows that the 1%, 0.5%, and 0.2% ACE flood hydrographs are similar, and the differences are peaks and volume flows. Table 5 shows the 3.5 days (84 hours) total volume flows and inundated areas for the three different flood frequencies. The inflow hydrograph duration is 3.5 day (84 hours) for all the simulations.

6.3.3 Scenario 3: Impingement & Piping Failure at Four Locations

The third scenario assumes that the levee fails at four different locations due to impingement and piping failures. The levee failure was assumed at four locations. Impingement failure was assumed to happen at three locations along the levee. One location of piping failure was also assumed. The failure locations were assumed based on the geotechnical study input. Plate 21 shows the simulated floodplain for the 1% ACE flood. The levee failure locations are also shown in the simulated floodplain maps. The simulated floodplain is similar to that of Plate 18, the total levee failure case (Scenario 2).

Plate 22 shows the simulated floodplain for the 0.5% ACE flood. Plate 23 shows the simulated floodplain for the 0.2% ACE flood. Comparisons between Plate 21, Plate 22, and Plate 23 show that the flooding areas are similar for the three flood frequencies. The differences are the flow depth. The results are also similar to the simulation results of the total levee failure case (Scenario 2). Table 6 shows the 3.5 days total volume flows and inundated areas for the three different flood frequencies. The simulated inundation areas for the impingement and piping failure are slightly larger than the corresponding areas simulated for the no levee case (Scenario 2). The major difference is in the lower end of the levee area.

6.3.4 Scenario 4: Impingement Failure near State Route 87 Bridge

The fourth scenario assumes that the levee fails due to impingement near the SR 87 Bridge area. The levee failure was assumed at only impingement failure point 1, close to SR 87. Plate 24 shows the simulated floodplain for the 1% ACE flood. As shown in the figure, the flood water flows through the narrow funnel area between SR 87 and I-40 into the city of Winslow. Through the I-40 bypass area, the floods flow further through I-40 to the north (downstream) and create a large flooding area.

As demonstrated in this simulation scenario, the levee breach failure in the upstream area will cause significant damage to the city of Winslow, Ames Acres, Bushman Acres, and other areas behind the levee.

6.3.5 Scenario 5: Impingement & Piping Failure Downstream of Bushman Acres

The fifth scenario assumes that the levee fails at three locations downstream of the Bushman Acres community. The levee failure was assumed at three locations downstream of the Bushman Acres area. Plate 25 shows the simulated floodplain for the 1% ACE flood and the levee failure locations. As shown in Plate 25, the flood water flows downstream toward the north. The city of Winslow, Bushman Acres, and other residential areas are not inundated by the floods.

As demonstrated in this simulation scenario, the levee breach failure in the downstream area of Bushman Acres will not cause damages to the city of Winslow, Ames Acres, Bushman Acres, and other business areas behind the levee.

6.4 Comparison of FLO-2D Results with HEC-RAS Results

The LCR Winslow study area is in an alluvial fan valley with a flat central area and gradually higher elevations at the east and west sides of the valley. The comparison of the HEC-RAS and FLO-2D models provides a better understanding of floodplains of the study area.

The HEC-RAS floodplains for the 4% ACE and 2% ACE floods are shown on Plate 11 and 12. Plates 13, 14, and 15 show the HEC-RAS 1% ACE, 0.5%, and 0.2% ACE floodplains, respectively. The comparisons of the HEC-RAS simulated 4% ACE and 2% ACE floodplains (Plates 11 and 12) with the FLO-2D simulated 4% ACE and 2% ACE floodplains (Plates 16 and 17) show that the results from the two model simulations are compatible and in agreement. For these two flood frequencies, the levee remained functional and the water flows were contained behind the levee.

A comparison of the HEC-RAS simulated 1% ACE flood (Plate 13) with the FLO-2D simulated 1% ACE floodplain (Plate 18) shows that the results from the two model simulated floodplains are very similar.

The comparisons between the HEC-RAS simulated 0.5% ACE and 0.2% ACE floodplains (Plates 14 and 15) with the FLO-2D simulated 0.5% ACE and 0.2% ACE floodplains (Plates 19 and 20) show that the results also align.

6.5 Summary of FLO-2D Results

As mentioned in Section 5.0, the floodplains simulated by the HEC-RAS model were based on the steady state flow rate boundary condition. FLO-2D is a volume conservation two-dimensional model and uses unsteady state flow hydrograph as input. The FLO-2D model simulations verify that the floodplains generated from the HEC-RAS simulations are consistent.

In addition to the baseline model simulations, the FLO-2D model provided three more levee failure case analyses. The impingement and piping failure at four possible locations demonstrated that it will cause almost the same inundation area as the total levee failure. The impingement levee failure near SR 87 will cause much more damage than the impingement levee failure downstream of the Bushman Acre area. As mentioned in Section 6.3 the FLO-2D simulations were used to complement the HEC-RAS model simulations. The FLO-2D simulations provide valuable input for the plan formulation process in the baseline study and the project alternative evaluations. In this study, the HEC-RAS results were used for the economic analysis (HEC-FDA). The HEC-RAS model was also used for the Risk & Uncertainty Analysis.

The 4% ACE and 2% ACE floods for the FLO-2D baseline condition do not show flooding along the Winslow Levee reach (Plates 16 and 17). The 1%, 0.5%, and 0.2% ACE floods do show flooding in the City of Winslow caused by failure of the Winslow Levee (see Plates 18, 19, and 20). For the 1%, 0.5%, and 0.2% ACE floods, maximum flood depths in the City of Winslow southwest from I-40 are approximately 7, 8, and 10 feet, respectively. Each of the three flood frequencies inundate the area around the wastewater treatment plant (WWTP) which has a ring levee surrounding it which was designed for the 1% ACE flood. Maximum flow depths near the WWTP are 3, 5, and 6 feet respectively for the 1%, 0.5%, and 0.2% ACE floods.

The baseline condition shows flooding in the Homolovi I Pueblo area. For Scenario 1 (levee remains – no failure), the 1% ACE flow depth is a maximum of 5.1 feet. For Scenario 3 (4 levee failure locations), the 1% ACE flow depth is a maximum of 4.4 feet. For Scenario 4 (levee failure upstream near State Route 87 Bridge), the 1% ACE flow depth is a maximum of 4.5 feet. For Scenario 5 (levee failure downstream of Bushman Acres), the 1% ACE flow depth is a maximum of 4.9 feet.

7.0 SEDIMENT TRANSPORT ANALYSIS

This section presents the sediment transport analyses for the Winslow Levee Study Reach of the LCR. The objective of the analyses is to identify sedimentation and erosion under the baseline condition. The channel geomorphology, the HEC-RAS sediment model development, and model simulations are presented in this section. See Attachment 2 for Sediment Gradations and Gradation Location Map. See Attachment 3 for a description on channel geomorphology.

7.1 Sediment Transport Model

HEC-RAS version 4.2 beta was used to conduct the numerical sediment transport modeling in this study at the recommendation of HEC. The objective of the sediment transport analysis is to identify baseline sediment conditions. The sediment transport analysis is conducted to establish channel configuration for future conditions based on sediment impacts. A baseline condition sediment transport model was created based on the baseline hydraulic model and was extended upstream about 10 miles to better represent the sediment transport of LCR.

Because measured water surface elevations were not available, calibration could not be performed due to the lack of historical data. Manning's roughness coefficients, detailed in Section 5.3.2, were used for the sediment transport model. Water surface elevations at hydraulic river station 5+00 from the hydraulic analysis were used for the rating curve and are provided in Table 7. Bridges were removed from the baseline condition hydraulic model for the sediment transport analysis for numerical stability.

7.2 Comparison of Survey Data

The sediment analysis was based on best available data, such as survey data from the USBR study in 2003 (Reference E) and the 2009 Winslow survey data provided by Navajo County (reference C) as well as technical manuals. The sediment transport modeling for the LCR provides insight to the current and future conditions of the river system, helping determine if areas are aggrading, degrading, or stable. The 2009 Winslow survey data was compared to the USBR survey (completed in 2000) at various cross-sections along the study reach. The cross-sections from the two surveys were analyzed to check for any aggradation or degradation along the reach over the 10-year period. The analysis showed that at the upstream portion of the study (from hydraulic river station 1300+00 to 750+00) the river floodplain experienced aggradation by as much as two feet, which occurred near hydraulic river station 1055+00. The comparison of the area upstream from the Winslow bridges (hydraulic river station 750+00 to 550+00) showed degradation by approximately 2 feet across the floodplain along this reach. The cross-sections near the bridges (hydraulic river station 550+00 to 480+00) showed both aggradation and degradation by as much as 1.5 feet along this reach of the LCR. The area downstream from the bridges (hydraulic river station 480+00 to 5+00) showed degradation by as much as 3.5 feet, which occurred at hydraulic river station 320+00. The HEC-RAS sediment model was compared to the aggradation and degradation trends that occurred over 10 years along the LCR.

7.3 Sediment Transport/Moveable Boundary Computations

The sediment transport model is designed to simulate long-term trends of scour and deposition in a stream channel. Sediment transport simulations were conducted for the 50%, 20%, 10%, 4%, 2%, 1%, 0.5%, and 0.2% ACE flood simulate a range of discharges that the sediment model would encounter during the movable bed simulations. The moveable bed limits were set for each cross-section based on the floodplain extents to allow for aggradation and degradation of the channel and floodplain.

7.4 Sediment Transport Functions

The Laursen-Copeland transport function and the Yang transport function were used for the baseline conditions sediment transport model. Yang's model was more appropriate for the analysis as it met the requirements for average channel velocity and sediment particle size better than Laursen-Copeland. It is the most appropriate model to use due to the sandy nature of the LCR bed material.

Laursen-Copeland was tested as one of the options as it covers a large range of grain sizes, and Yang's model was tested based on input bed material gradations. The average bed elevation difference for the 1% ACE flood model run using Laursen-Copeland was aggradation of 1.2 feet to degradation of 4.49 ft. The average bed elevation difference for the 1% ACE flood model using Yang was aggradation of 3.95 feet to degradation of 2.75 feet.

7.5 Bed Sediment Characteristics

Bed sediment characteristics and sample locations were provided by the USBR's sediment report from 2003 (Reference E). Ten locations from that report were used for sediment sampling within the project area, and gradation curves were developed at these locations. Those USBR samples (4, 6, 7, 9, 10, 12, 18, 19, 21, & 25) are shown in Attachment 2. Additionally, from hydraulic river station 1130+00 to 525+82.13, an average bed gradation from USBR #18 and USBR #19 was used in the model. USBR #18 is just downstream from Joseph City, AZ, and USBR #19 is located between the SR 87 and BNSF Railroad Bridges. Samples 4, 12, 6, 7, 10, and 9 from the USBR are located at the downstream portion of the reach. Sampling sites were located in natural channel areas, and samples were collected from 0 to 2 feet. Laboratory grain-size analyses were performed on the samples. The bed gradation data was entered in to the HEC-RAS 4.2 sediment data file, and the *interpolate gradation* option within HEC-RAS was utilized to determine the sediment for the remaining stations between the gradations from hydraulic river station 525+82.15 to 5+00. Sediment gradations and sample locations are shown in Attachment 2.

7.6 Inflowing Sediment Rating Curve

An equilibrium bed material load was used to determine the inflowing sediment loads. Equilibrium load is determined by transport capacity. Sediment transport capacity is determined at each time step at the specified cross-section, and is used as the sediment inflow. Since load is set equal to capacity for each grain size, there will be no aggradation or degradation at the upstream cross-section.

7.7 Movable Bed Limits

In general, sediment dynamics tend to be more significant within the active channel, where the bed can either degrade or aggrade in response to erosion or deposition. The overbank areas tend to be more stable and normally are free of erosion, but can experience deposition. The moveable bed limits were set to include part of the floodplain. A bed sediment depth of 8 feet was set as an initial condition. During sediment transport analysis, the largest degradation is approximately 3.2 feet for the 1% ACE flood, which is less than the initial moveable bed limit of 8 feet.

7.8 Hydrology

Simulations were performed with the peak discharge rates based on the HEC-RAS hydraulic results. Inflow hydrographs (See Section 4.2) at the upstream boundary of the channel are required. In order to preserve the peak flow and volumes of the flooding events, the balanced hydrographs were calculated for the simulation. In addition, balanced hydrographs for 50%, 20%, 10%, 4%, 2%, 1%, 0.5%, and 0.2% ACE floods were used to simulate the impact that such events would have on the channel.

7.9 Local Scour

The sediment transport analysis was conducted to determine the aggradation and degradation for the various discharge frequency events. The analysis detailed in previous sections does include local scour. Based on the 1980 LCR *Feasibility Report* (Reference P), the design scour depth of the Winslow Levee varies from 10 to 15 feet for the reaches where the river is susceptible to sharp bends and impingement. A scour depth of 5 feet was recommended for the reaches of the levee that were not in contact with the main channel of the river. For the alternatives described in this appendix, 15 feet was used as the scour depth for the levee design along the entire length because impingement locations have historically moved and to be conservative. A scour analysis is recommended for the tentatively selected plan to verify the previous analysis from 1980.

7.10 Results

The following results are for the 1% ACE flood. The results for the other flood frequencies are similar. The upstream end of the LCR from hydraulic river station 1070+00 to 1005+00 experienced degradation up to 2.2 feet. From 1005+00 to 760+00, the channel generally experiences aggradation up to 1.1 feet. The channel experiences degradation near the bridge area from hydraulic river station 565+00 to 490+00 of up to 3.2 feet. The degradation can be attributed to the bridges constricting flow at greater discharge frequencies. Downstream of the bridge area, the channel experiences mostly aggradation of up to 0.95 feet from hydraulic river station 490+00 to 375+00. Figures 5 through 12 show the channel invert change for the 50%, 20%, 10%, 4%, 2%, 1%, 0.5%, and 0.2% ACE floods.

No data was used for calibration. The sediment analysis was based on best available data, such as survey data from the USBR study in 2003 and the 2009 Winslow survey completed by Navajo County as well as technical manuals. The results of the sediment study will be used to help evaluate alternatives in the next phase of the Winslow study.

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8.0 RISK AND UNCERTAINTY ANALYSIS

A Risk and Uncertainty (R&U) analysis was performed for the baseline condition using procedures in EM 1100-2-1619, *Risk-Based Analysis for Flood Damage Reduction Studies* (Reference Q) and ER 1105-2-101, *Risk Analysis for Flood Damage Reduction Studies* (Reference R). These documents discuss uncertainties associated with various elements of the design, including discharge-probability, stage-discharge, and structural and geotechnical performances, as well as methodologies to quantify these uncertainties. The current USACE methodology for the R&U analysis does not consider structural and geotechnical uncertainties; thus, the analysis for this study was conducted for only discharge-probability and stage-discharge functions. In addition, uncertainty associated with Manning's "n" values was incorporated using the standard deviation of error in the stage-discharge functions.

8.1 Sources of Uncertainties – Discharge Probability

For a flood or storm event with a given probability of occurrence, there is uncertainty regarding the discharge at specific locations along the study reach. The reliability of discharge/probability estimates is directly linked to the available historical record of stream gage data. In cases where records are short or incomplete, the uncertainty tends to be large. To address this uncertainty, an analytical or graphical method is typically used to determine statistical distributions of discharge for a range of probabilities at key locations in the study area.

The stream gage for the LCR near Winslow has only 7 years of data (2002 - 2008). Therefore, stream gage records for the LCR near Joseph City, Holbrook, and Grand Falls were also collected. It is important to note that the stream gages near Joseph City (09397300) and at Holbrook (09397000) are separated by a distance about 7 - 8 miles, and their drainage areas are 11,462 square miles and 12,384 square miles, respectively. One gage has continuous data from 1950-1972, and the other gage has continuous data from 1971-2008. Therefore, these two stream gage records are combined together by applying the drainage-area-ratio methodology (Reference I). The combined stream gage records have 61 years of data. The total drainage area is 16,192 square miles. Table 8 shows the expected and computed peak discharges with confidence limits for the LCR at Winslow. The computed discharge for the 50%, 20%, 10%, 4%, 2%, 1%, 0.5%, and 0.2% ACE floods is provided in Table 2. The computed discharge for the 99% ACE flood is 1,620 cfs.

8.2 Sources of Uncertainties – Stage Discharge

For a given discharge, there is uncertainty regarding the water surface elevation at a given location. Factors contributing to this uncertainty include bed forms, water temperature, debris or other obstructions, unsteady flow effects, variation in hydraulic roughness with season, sediment transport, channel scour or deposition, and changes in channel shape during or as a result of flood events, among other factors. To address this uncertainty, estimates of the standard deviation of error about the predicted stages at key locations were made.

Total stage uncertainty is a function of model uncertainty (S_{model}) and natural uncertainty ($S_{natural}$).

8.2.1 Natural Uncertainty

Natural uncertainty is a function of four parameters: watercourse bed composition, drainage area, 1% ACE peak discharge, and stage range.

Watercourse Bed Composition (Bed Identifier)
With respect to the water course bed composition factor, information in “Table 5-1” of EM 1110-2-1619 was utilized (see Table 9 of this report). A higher value relates to higher “mobility” of the bed material. (Note this is completely independent of the smoothness of the bed material). Manning’s n value variation is a parameter of S_{model} (See Section 8.2.2). For the bed material, the user evaluates the probability that the bottom topography will remain unchanged over time. A less “mobile” material will be more able to resist scour and erosion. Since the evaluation reach invert is comprised of sands, the bed composition factor corresponds to 4.

Drainage Area
As discussed in the USACE hydrology appendix, the Winslow area has a drainage area of approximately 16,192 square miles.

The 1% ACE Peak Discharge
Through the evaluation reach, the 1% ACE peak discharge used for the analysis is 69,200 cfs for the LCR at Winslow (Refer to Hydrology Appendix).

Stage Range
Range is defined as the maximum predicted or observed range of stage on the watercourse. The minimum flow in the river is set to zero; therefore, the minimum water surface elevation is equal to the invert elevation at any location. In a theoretical worst case scenario, the water surface could rise to the height of the levee and then by some additional value while overflowing. For this evaluation, the height of the levees plus one foot was determined to be the maximum water surface elevation at any given cross-section river station.

The four parameters listed above serve as inputs for the equation below, which yields natural uncertainty. As explained in EM 1110-2-1619, this equation is written to use metric units of measure and therefore requires conversion before calculating.

$$S_{tnatural} = [\ 0.07208 + 0.04936\ I_{bed} - 2.2626 \times 10^{-7}\ A_{basin} + 0.02164\ H_{range} + 1.4194 \times 10^{-5}\ Q_{100}\ ]^2$$

I_{bed} = stream bed identifier for the size of the bed material (dimensionless constant), which controls flow in the reach of interest
A_{basin} = drainage basin area in square kilometers
H_{range} = maximum expected or observed range in stage in meters
Q_{100} = peak discharge of the (1% ACE) flood in cubic meters per second

Since an HEC-RAS model was used to obtain invert and levee elevation data for each designated cross-section. The $S_{natural}$ value was determined for each cross-section.

8.2.2 Model Uncertainty

As defined in EM 1110-2-1619, model uncertainty is associated with the accuracy of the Manning's n-values used in the model of the watercourse. Because the n-value is not a measurable quantity, there is some inherent uncertainty with the n-values used in a computer model or a mathematical calculation. As mentioned, the Manning's n-value determination is not exact.

To calculate the model uncertainty, two modified geometries for the evaluation reach were created in HEC-RAS. The Manning's n-values were reduced by 20% for one of the geometry files and increased by 20% for the other geometry file. Manning's n-values were varied by 20% based on reasonable engineering assessment to develop upper and lower bounds of stage as described in EM 1110-2-1619, Section 5-7.

The value of model uncertainty is the standard deviation of the variation in water surface elevations between the "best case" and "worst case" geometries. A steady state analysis was conducted for each geometry file using HEC-RAS. The output results from both iterations were then displayed on a spreadsheet with a focus on determining the water surface elevation at each cross-section within the evaluation reach. Finally, the water surface values were averaged to determine an E_{mean} for each damage sub-reach. The sub-reaches used for this analysis are Reaches 1 and Reach 2 as shown in Plate 26. The deviation in the water surface profiles was then calculated using the equation below.

$$S_{\mathrm{model}} = \frac{E_{mean}}{4}$$

Where E_{mean} = mean difference between the upper and lower limits of the calculated stage

8.2.3 Total Uncertainty

Model and natural uncertainty are related using Eqn. 5-6 of EM 1110-2-1619 to calculate the total uncertainty at each cross-section for each damage sub reach. The total uncertainty (S_{total}) value at each station within each sub-reach was then averaged to determine the total uncertainty for the respective damage sub-reach. Because the HEC-FDA program uses one designated index station within each sub-reach, the natural uncertainty for that specific station was averaged with the model uncertainty for the encompassing sub-reach to calculate the total uncertainty at that index station applicable to its damage sub-reach (defined below). The index location for the evaluation reach is specified to aggregate stage-damage functions with uncertainty for flood damage analysis calculations. For this analysis the index location was set at the cross-section location with the least freeboard or areas where the levee has is susceptible to overtopping or failure. The total standard deviation of uncertainty calculation is summarized in Table 10.

$$S_t = \sqrt{S^2_{natural} + S^2_{\mathrm{model}}}$$

Where S_t = total standard deviation of uncertainty
$S_{natural}$ = standard deviation of uncertainty as a function of pertinent natural physical characteristics of the watershed and conveyance
S_{model} = standard deviation of uncertainty of computed water surface data using mathematical models

8.3 Geotechnical Evaluation of Levee Fragility

8.3.1 Overview

A Geotechnical Engineering analysis was conducted by Los Angeles and San Francisco Districts of the U.S. Army Corps of Engineers. The purpose of the geotechnical analysis was to evaluate the expected geotechnical performance of the Winslow and Ruby Wash Diversion levees, located near Winslow, Arizona. The evaluation was used in performing an economic cost/benefit analysis to determine if a flood risk management project is feasible in accordance with EM 1110-2-1619, Risk Based Analysis for Flood Damage Reduction Studies. See Attachment 4 for the *Geotechnical Evaluation of Levee Fragility* report completed in 2012.

8.3.2 Levee Fragility and Probability of Unsatisfactory Performance

As discussed in ETL 1110-2-556 probabilistic engineering analysis is a complex and immature field in geotechnical engineering, and the results of this analysis should be used and interpreted with care.

Corps planning has adopted a risk and uncertainty modeling approach, requiring that that the geotechnical performance of the levee be considered when determining cost/benefit ratios. The geotechnical performance is stochastically incorporated into the economics by the use of levee fragility curves that express the probability that the levee will have unsatisfactory performance for a given river stage.

Typically, fragility curves are used in the economic analysis in a joint probability approach combining event frequency and probability of unsatisfactory performance, such that the damages for a given event are effectively scaled by the probability of unsatisfactory performance. The damages for all possible events are determined and annualized to compute estimated annual damages. Probability of Unsatisfactory Performance (P_u) is used to define fragility curves. P_u does not directly describe the probability that the levee will catastrophically fail under a given load, but rather describes the probability that ground conditions exist that would result in a limit state (factor of safety =1.0) being exceeded under the given load for a certain set of assumptions.

Levee failure modes include underseepage (when landside slope stability factor of safety is reduced below 1.0) or when an erosion progression occurs. The “weak link” for each economic area was chosen to calculate damages for the without project condition.

8.3.3 Selection of Index Locations

The levee, for the purposes of this analysis, was divided into reaches. A reach was defined as a segment of levee which, if a breach were to occur at any point within that segment, would likely result in similar damages. An index point was defined as a critical cross section at a specific station within each reach. The project geotechnical team considered levee geometry, geotechnical conditions, hydraulic loading, past performance and potential economic consequences in selecting index points for levee fragility monitoring. If conditions did not readily allow for determining between two locations in a reach, which was likely to have worse geotechnical performance, both were evaluated. The most fragile index point was chosen to represent the levee in that reach. Three reaches were defined and four index points were selected for geotechnical fragility evaluation.

The four index locations selected for the geotechnical levee fragility analysis were:

1. Ruby Wash Diversion Levee (RWDL) hydraulic river station 4+95 – equivalent to Winslow Levee (WL) hydraulic river station 535+00 (upstream from the BNSF Railroad Bridge).
2. Winslow Levee hydraulic river station 515+00 (downstream from the State Route 87 Bridge and upstream from the Interstate 40 Bridges).
3. Winslow Levee hydraulic river station 370+00 (located near the southern impingement location due west of the Homolovi I Pueblo).
4. Winslow Levee hydraulic river station 290+00 (located near the northern impingement location approximately 12,000 feet downstream (north) from the Homolovi I Pueblo).

See Figure 4 on page 12 of Attachment 4 for index point locations.

8.3.4 Evaluation of Index Locations

Table 11 shows the combined probability of unsatisfactory performance for each of the levee locations. Table 11 shows whether the water surface elevation is above (positive number) or below (negative number) the top of levee for the WSE values selected in the delineation of the levee fragility curves. Table 12 shows probability of unsatisfactory performance vs frequency and discharge events.

At index location 1 (RWDL 4+95/ WL 535+00), the probability of unsatisfactory performance begins at water surface elevation 4862 feet (10% ACE flood) which is 4.7 feet below the top of levee elevation. The probability of unsatisfactory performance increases to 0.16 for the 4% ACE flood, 0.31 for the 2% ACE flood, and 1.0 at WSE 4868 feet (which is between a 2% and 1% ACE flood) and approximately a foot above the top of levee elevation.

At index location 2 (WL 515+00), the probability of unsatisfactory performance begins at water surface elevation 4859.5 feet (between a 10% and 4% ACE flood) which is 5.5 feet below the top of levee elevation. The probability of unsatisfactory performance increases to 0.01 for the 4% ACE flood, 0.03 for the 2% ACE flood, 0.08 for the 1% ACE flood, and 1.0 at WSE 4866 feet (which is between a 1% and 0.5% ACE flood) and is 1 foot above the top of levee elevation.

At index location 3 (WL 370+00), the probability of unsatisfactory performance begins at water surface elevation 4849 feet (between a 10% and 4% ACE flood) which is 6.4 feet below the top of levee elevation. The probability of unsatisfactory performance increases to 0.01 for the 4% ACE flood, 0.03 for the 2 % ACE flood, 0.07 for the 1% ACE flood, 0.16 for the 0.5% ACE flood, 0.20 for the 0.2 % ACE flood, and 0.50 at WSE 4857 feet (which is greater than a 0.2% ACE flood) and is approximately 1.5 feet above the top of levee elevation.

At index location 4 (WL 290+00), the probability of unsatisfactory performance begins at water surface elevation 4840 feet (less than a 50% ACE flood) which is 12.6 feet below the top of levee elevation. The probability of unsatisfactory performance increases to 0.23 for the 4% ACE flood, 0.38 for the 2 % ACE flood, 0.57 for the 1% ACE flood, 0.78 for the 0.5% ACE flood, 0.92 for the 0.2 % ACE flood, and 1.0 at WSE 4852 feet (which is greater than a 0.2% ACE flood) and is approximately 0.6 feet below the top of levee elevation.

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9.0 ALTERNATIVES ANALYSIS OVERVIEW

This section discusses the with-project hydraulic analysis, provides an overview of the plan formulation charette process, and briefly describes the alternatives that were developed for this study. The plan formulation charette resulted in formulation of seven alternatives, five of which were carried forward. Six action alternatives were eventually developed to address the flooding concerns in the Winslow area. Later in the plan formulation process, four additional action alternatives were developed. Hydraulic analysis was completed to analyze six conveyance measures with the intention of increasing flow conveyance under the BNSF Railroad Bridge to decrease the flooding impacts at this location. The selected conveyance measure was included in the four additional structural alternatives described in Section 10.11.

9.1 Plan Formulation Charette and General Objectives

A plan formulation charette workshop was held in Winslow, Arizona on 29-31 May 2012. The primary purpose of the charette was to use this collaborative process to expedite plan formulation for the preliminary array of alternatives. The charette was a forum to discuss problems, opportunities, and constraints for use in the plan formulation as well as to identify study objectives, alternatives, and associated measures for further analysis.

The main objectives of the study are to reduce the public safety and health risk to the Winslow community due to historical flooding and to reduce the risk of damages due to flooding in the City of Winslow and surrounding areas. The local objective is to provide a levee system that is capable of being accredited by FEMA for the 1% ACE or “100-year” flood event. The interest in supporting such a plan is to eliminate the City of Winslow and surrounding communities from the “100-year floodplain.”

Teams in the charette were asked to identify both structural and nonstructural measures to address the problems and opportunities and to meet the objectives. The measures developed looked at nonstructural measures such as elevating homes, in-channel measures such as channelization, and levee measures such as new and setback levee segments.

After measures were developed, preliminary alternatives were formulated based on the structural and nonstructural measures. Seven preliminary action alternatives were developed during the charette, including rehabilitation of the Winslow Levee, new levee construction near existing alignment, realignment and setback of the levee, construction of a new levee parallel to I-40 with floodgates at underpasses, upstream detention, channelization of the Little Colorado River, and nonstructural measures north of I-40. The seven preliminary alternatives were evaluated based on completeness, effectiveness, efficiency, acceptability, and preliminary cost. A summary of the measures and alternatives developed during the charette can be found in the Little Colorado River at Winslow, Arizona Post Charette Report (Reference S).

Two alternatives were screened out during the charette including upstream detention and channelization of the Little Colorado River. The alternatives were screened out due to various reasons, including environmental impacts from channelization or dam construction and increased flooding downstream.

Five alternatives were carried forward including rehabilitation of the Winslow Levee, new levee construction near existing alignment, realignment and setback of the levee, nonstructural measures, and construction of a new levee parallel to I-40 with floodgates at underpasses. The four structural alternatives carried forward from the charette included various combinations of either levee rehabilitation and/or new levee alignments and were carried forward for hydraulic analysis (Alternatives 1.1, 3.1, 8, and 10).

The nonstructural alternative included nonstructural measures north of I-40 without any levee improvements (Alternatives 7). The nonstructural measures that were carried forward from the plan formulation charette included the following: an improved flood warning system and/or elevating homes.

9.2 With-Project Alternatives

The with-project alternatives that were analyzed as part of this study are listed below and described in Section 10.0. The levee alternatives (Alternatives 1.1, 3.1, 8, and 10) were designed to provide 1% ACE flood protection for the City of Winslow with a levee freeboard of 3 feet in lieu of risk and uncertainty at this point and to meet the objectives regarding flood risk reduction. To compare the four levee alternatives, 1% ACE flood protection plus three feet freeboard was selected for the levee heights. Levee optimization was completed for Alternative 10. The alternatives for this study include the following:

- Alternative 1.1 – Rebuild Levees, New Levee Parallel to I-40, Conveyance Improvements
- Alternative 3.1 - Setback Levees, Rebuild Levees, New Levee Parallel to I-40, Conveyance Improvements
- Alternative 7 – Nonstructural Measures North of I-40, No Levee or Conveyance Improvements
- Alternative 8 – Rebuild Levees, Setback Levee, New Levee Parallel to I-40, Conveyance Improvements
- Alternative 9 – Levee Increment 1 - Rebuild RWDL at Existing Height, No Conveyance Improvements, Nonstructural Measures North of I-40
- Alternative 10 – Levee Increments 1 & 2, Rebuild & Setback Levees (ending at hydraulic river station 320+00), Conveyance Improvements and Nonstructural Measures North of I-40 (Note: Alternative 10 includes 4 optimization alternatives discussed in Section 10.11)
- Alternative 11: No Action

All alternatives include non-structural measure of a flood warning system. Four of the six with-project alternatives include “conveyance improvements” under BNSF Railroad Bridge. Additional hydraulic analysis was completed to evaluate six conveyance measures that would decrease the water surface elevation (WSE) at the BNSF Railroad Bridge in an attempt to prevent overtopping of the railroad bridge and the Winslow Levee. Section 10.11 provides more information.

10.0 HYDRAULIC ANALYSIS OF WITH-PROJECT ALTERNATIVES

10.1 Introduction to Modeling Procedures

This section describes the hydraulic analysis for the present with-project condition. A one dimensional steady flow model was developed using HEC-RAS version 4.2 beta July 2013. It was also used to run the baseline condition sediment transport analysis. HEC recommended that HEC-RAS version 4.2 beta be used for the sediment transport analysis, since the beta version included a few bug fixes from version 4.1.

HEC-GeoRAS was applied to assist the HEC-RAS model development. Water surface profiles were developed for the 50%, 20%, 10%, 4%, 2%, 1%, 0.5%, and 0.2% ACE floods for the six alternatives and the four optimization alternatives described in the following sections.

The HEC-RAS model parameters in regards to cross-sections, Manning's n-values, bridges, flow regime, and boundary condition are the same as described in Section 5.3. The assigned Manning's n-values for the with-project model do vary from the without-project model in some locations, such as in locations that were previously on the land side of the levee but for an alternative now reside on the river side. The Manning's n-value in these locations was updated to match current designations on the river side of the levee in the surrounding area.

10.2 Hydraulic Design

10.2.1 Conveyance Improvements at BNSF Railroad Bridge

Four of the six with-project alternatives mentioned above include conveyance improvements under BNSF Railroad Bridge. The four optimization alternatives also include the conveyance improvements. Hydraulic analysis was completed to evaluate six conveyance measures that would decrease the WSE at the BNSF Railroad Bridge in an attempt to prevent overtopping of the railroad bridge and the Winslow Levee. The conveyance measures discussed included excavating and widening the channel, removing saltcedar, lining a portion of the river bottom with concrete, extending the railroad bridge opening, and installing culverts on either side of the railroad bridge. See Attachment 5 for a detailed description of the conveyance measures considered.

HEC-RAS was used to analyze the various measures, and each measure includes saltcedar removal in some capacity. The roughness of the saltcedar is so high, without its removal, water from the 1% ACE flood event would not be able to convey under the BNSF Railroad Bridge, even if other conveyance improvements were implemented. Extending the bridge opening and installing culverts were two measures that were considered but not modeled as part of this analysis due to BNSF Railroad concerns. The purpose of the conveyance measures analysis was to increase capacity at the BNSF Railroad Bridge in order to decrease the WSE at the Winslow Levee upstream from the bridge. The measures were intended to convey the 1% ACE flood (69,200 cfs).

The selected conveyance measure was Conveyance Measure C. For the additional Conveyance Measures considered, see the attached Attachment 5. Measure C includes the excavation and widening of the channel bottom from approximately 1,000 feet upstream from the BNSF

Railroad Bridge at hydraulic river station 540+00 to approximately 1,000 feet downstream from the SR 87 Bridge at hydraulic river station 515+00. This reach extends approximately 2,500 feet and includes excavation and widening of the LCR. No attempt was made to deepen the channel throughout the reach. The excavation and widening of the channel begins at hydraulic river station 540+00. The width of the excavated channel varies from 200 feet at hydraulic river station 540+00 to 650 feet at the BNSF Railroad Bridge (529+39.3). Downstream from the BNSF Bridge, the channel maintains a width of approximately 650 feet through the SR 87 Bridge. Downstream from the SR 87 Bridge, the channel width varies from 650 feet to 200 feet at hydraulic river station 515+00. The excavation area is approximately 26 acres. This measure includes the removal of approximately 96 acres of saltcedar throughout the reach.

Analysis regarding the expected geomorphic response was not conducted for the channel modification, and it is recommended that geomorphic analysis be conducted during the PED phase.

10.2.2 Levee Height Overview

The structural alternatives (Alternatives 1.1, 3.1, 8, and 10) include rebuilding levee segments or constructing new levee segments. The levee heights for these alternatives are based on the water surface profile for the 1% ACE flood. Engineer Circular 1110-2-6067 (Reference J) was used to determine the required levee height above the water surface (freeboard). From R&U analysis, three feet of additional levee height was used to increase the assurance that the specified flood could be contained. The alternatives discussed in this section use three feet of additional levee height for the hydraulic analyses; however, further economic analysis indicated that an additional levee height of 3.3 feet would be required for Alternative 10.1 to meet the requirement of 90% CNP. Table 13 provides the average and maximum change in levee heights for reaches along the levee system for each of the alternatives.

The increase in levee height is the difference in the levee height for each alternative compared to the existing levee height. All stationing provided in the table corresponds to the HEC-RAS model stationing. Alternative 7 did not include additional levee height as this alternative dos not include any levee improvements. Alternative 9 includes rebuilding the RWDL at existing height and does not include additional levee height based on risk and uncertainty analysis.

10.2.3 Slope Protection Overview

Riprap is broken stone that interlocks when properly placed to resist erosion by rising river flows and utilizing strength from its mass and its interlocking ability. Grouted stone is similar to riprap but its strength and erosion resistance comes from the grout and stone placement. The stone used for grouted stone protection is sized and placed differently than riprap to assure interconnected void space, which allows extensive grout penetration.

Chapter 3, Riprap Protection, from Engineer Manual 1110-2-1601 (Reference T), *Hydraulic Design of Flood Control Channels*, was used in the riprap protection analysis. CHANLPRO, Version 2.0 was also used in the analysis.

Slope protection recommendations along the river side of the levee are provided in Table 14. The table indicates whether riprap, soil cement, or grouted stone is recommended for each reach. The reaches that specify soil cement slope protection can have grouted stone slope protection.

10.3 With-Project Alternatives

The six original with-project condition action alternatives were considered for this analysis. All alternatives described below were modeled using HEC-RAS with the exception of Alternative 11 which is the "No Action" Alternative. Alternative 11 is the same as the baseline condition.

The following sections describe the with-project alternatives. Some of the alternatives include raising the levee (increasing the levee elevations above the existing levee elevations). A detailed scour analysis has not been completed, so the current design assumes scour depths and has toe-down depths that range from 10-15 feet.

10.4 Alternative 1.1 – Rebuild Levees, New Levee Parallel to I-40, Conveyance Improvements

10.4.1 Alternative Description

Alternative 1.1 includes rebuilding the Winslow Levee and the eastern end of the Ruby Wash Diversion Levee along their current alignments, constructing a new levee parallel to I-40, and improving the conveyance under the BNSF Railroad Bridge. This alternative provides flood damage reduction to the City of Winslow, Bushman Acres, and the area near the WWTP. It also provides flood damage reduction to I-40, SR 87, and the BNSF Railroad west of the LCR. Plate 27 shows the features of Alternative 1.1 and the 1% ACE floodplain.

10.4.2 Slope Protection

Table 14 presents the slope protection recommendations for this alternative which include riprap and soil cement/grouted stone. The slope protection recommendations for Alternative 1.1 are separated into 8 reaches. The riprap slope protection includes six reaches with 24-inch or 42-inch riprap. There are two reaches near the south and north impingement locations that require soil cement or grouted stone. Plate 4 shows two of the three impingement locations along the Winslow Levee.

10.5 Alternative 3.1 – Setback Levees, Rebuild Levees, New Levee Parallel to I-40, Conveyance Improvements

10.5.1 Alternative Description

Alternative 3.1 includes rebuilding part of the Winslow Levee along its current alignment, setting back part of the Winslow Levee, removing the original Winslow Levee in the setback areas, rebuilding the eastern end of the RWDL along its current alignment, constructing a new levee parallel to I-40, and improving conveyance under the BNSF Railroad Bridge. This alternative provides flood damage reduction to the City of Winslow, Bushman Acres, and the area near the WWTP. It also provides flood damage reduction to I-40, SR 87, and the BNSF Railroad west of the LCR. This alternative provides similar protection to the City of Winslow as Alternative 1.1; however, it does flood additional area due to the setback levees. Plate 28 shows the features of Alternative 3.1 and the 1% ACE floodplain.

10.5.2 Slope Protection

Table 14 presents the slope protection recommendations for this alternative which include riprap and soil cement/grouted stone. The slope protection recommendations for Alternative 3.1 are separated into 7 reaches. The riprap slope protection includes six reaches with 24-inch or 42-inch riprap. There are two reaches near the south and north impingement locations that require soil cement or grouted stone.

10.6 Alternative 7 – Nonstructural Measures North of I-40; No Levee or Conveyance Improvements

10.6.1 Alternative Description

Alternative 7 employs nonstructural flood risk management measures for residences located north of I-40 only. The nonstructural measures include: an improved flood warning system and raising structures.

This alternative does not include any levee or conveyance improvements. Nonstructural measures would be used to reduce the risk of flooding for this alternative. Plate 29 shows the features of Alternative 7 and the 1% ACE floodplain.

For this alternative, the Winslow Levee is not raised or rebuilt and does not include any additional slope protection recommendations. This alternative does not include any improvements to the RWDL.

10.7 Alternative 8 – Rebuild Levees, Setback Levee, New Levee Parallel to I-40, Conveyance Improvements

10.7.1 Alternative Description

Alternative 8 includes rebuilding most of the Winslow Levee along its current alignment, setting back a short segment of the Winslow Levee directly west of the Homolovi I Pueblo, removing the original Winslow Levee in the setback area, rebuilding the eastern end of the RWDL, constructing a new levee parallel to I-40, and improving conveyance under the BNSF Railroad Bridge. This alternative was specifically designed to provide adequate freeboard to meet FEMA levee accreditation requirements. This alternative provides flood damage reduction to the City of Winslow, Bushman Acres, and Ames Acres. It also provides flood damage reduction to I-40, SR 87, and the BNSF Railroad west of the LCR. This alternative provides similar protection to the City of Winslow as Alternative 1.1; however, it does flood additional area due to the setback levee near the southern impingement location. Plate 30 shows the features of Alternative 8 and the 1% ACE floodplain.

10.7.2 Slope Protection

The slope protection for this alternative is similar to Alternative 1.1.

10.8 Alternative 9 – Levee Increment 1 – Rebuild RWDL at Existing Height, No Conveyance Improvements, Nonstructural Improvements North of Interstate 40

10.8.1 Alternative Description

This alternative includes rebuilding the eastern end of the RWDL at its existing height without improvements to the Winslow Levee, as well as employing nonstructural flood risk management measures for residences located north of I-40. Nonstructural measures are described in Section 10.6.1. This alternative would reduce the risk of flooding for floods up to the 2.7% ACE flood (36-year) discharge of 44,780 cfs. It does not include any conveyance improvements. Nonstructural measures would be used to reduce the risk of flooding for this alternative. Plate 31 shows the features of Alternative 9 and the 1% ACE floodplain.

For this alternative, the Winslow Levee was not raised or rebuilt. However, this alternative does include rebuilding the RWDL at existing height.

10.9 Alternative 10 – Levee Increments 1&2, Rebuild and Setback Winslow Levee, Rebuild RWDL, and Conveyance Improvements, and Nonstructural Measures North of I-40

10.9.1 Alternative Description

Alternative 10 includes rebuilding the Winslow Levee from the RWDL downstream to a point 0.8 miles north of North Road (hydraulic river station 320+00), no improvements to the Winslow Levee downstream of hydraulic river station 320+00 (as part of the federal project), setting back a short segment of the Winslow Levee across the LCR from the Homolovi I Pueblo, removing the original Winslow Levee in the setback area, rebuilding the eastern end of the RWDL, constructing a new levee parallel to I-40, improving conveyance under the BNSF Railroad Bridge, and employing nonstructural measures for residences downstream of North Road. Alternative 10 would provide structural measures to address the flood risk for the most densely developed portions of Winslow, with the use of nonstructural measures to reduce the risk further downstream. Plate 32 shows the features of Alternative 10 and the 1% ACE floodplain.

10.9.2 Slope Protection

Table 14 presents the slope protection recommendations for this alternative which include riprap and soil cement/grouted stone. The slope protection recommendations for Alternative 10 are separated into 6 reaches. The riprap slope protection includes five reaches with 24-inch or 42-inch riprap. There is one reach near the south river impingement location that requires soil cement or grouted stone.

10.10 Alternative 11 – No Action

The no action alternative is synonymous with the without-project baseline condition. No federal action would be undertaken to address the flood risk for the Winslow community. With the "No Action Alternative", the flood risk in the Winslow area is expected to remain essentially unchanged over the next 50 years.

10.11 Optimization of Alternative 10

Optimization of Alternative 10 was performed using 4%, 2%, 1%, and 0.5% ACE floods. The purpose of the optimization was to select the alternative and design frequency that provides the maximum benefit with consideration of the non-federal sponsor's preferences. Alternative 10.1 (1% ACE flood) was selected as the tentatively selected plan even though Alternative 10.4 (0.5% ACE flood) has greater net benefits. Alternative 10.1 provides the non-federal sponsor's desired maximum level of protection, features levee improvements designed to meet FEMA's flood insurance requirements, and has a greater net benefit than Alternatives 10.2 and 10.3.

Analysis was completed for the optimization of the Winslow Levee height for with-project Alternative 10. In addition, the elevation of residences was screened out for these optimized alternatives, since that measure was determined not efficient through later additional calculations. The additional hydraulic analysis was completed using Alternative 10 as a basis for design and included removing the nonstructural measures from the design. The following alternatives were developed during the optimization of levee heights for the respective flood frequencies.

a. Alternative 10.1 – 1% ACE Flood
b. Alternative 10.2 – 4% ACE Flood
c. Alternative 10.3 – 2% ACE Flood
d. Alternative 10.4 – 0.5% ACE Flood

Each optimization alternative resulted in a different water surface profile which was used to obtain levee heights for the Winslow Levee and RWDL. The levee heights for the Winslow Levee and RWDL for were set based on water surface profile computations using HEC-RAS. Three feet of additional levee height is included for each of the optimization alternatives to increase the assurance that the specified flood could be contained based on R&U analysis for Alternatives 10.1, 10.2, 10.3, and 10.4. The additional levee height that would be required for Alternative 10.1 is 3.3 feet based on economic analysis in order to get a CNP of 90%.

Table 13 provides the average and maximum change in levee heights for reaches along the levee system for each of the alternatives.

10.11.1 Risk and Uncertainty

R&U analyses were completed for each of the optimization alternatives and interior-exterior functions were provided to Economics Section of the Planning Division for USACE for analysis. Interior water surface elevations correspond to the water surface on the land side of the levee when the levee becomes overtopped or fails. Exterior water surface elevations correspond to the water surface on the river side of the levee before overtopping or failure.

R&U analyses were performed for Alternatives 10.1, 10.2, 10.3, and 10.4 using procedures in EM 1100-2-1619 (Reference Q) and ER 1105-2-101 (Reference R). These documents discuss uncertainties associated with various elements of the design, including discharge-probability, stage-discharge, and structural and geotechnical performances, as well as methodologies to quantify these uncertainties. The R&U procedures for the optimization alternatives are consistent with the procedures discussed in Section 8 of this appendix.

10.11.2 Alternative 10.1 – Levee Height based on 1% ACE

The levee heights for Alternative 10.1 are based on the water surface profiles for the 1% ACE flood. Alternative 10.1 includes the conveyance measure from Alternative 10 (described in Section 10.2.1). It includes excavation and widening of the channel bottom from approximately 1,000 feet upstream from the BNSF Railroad Bridge at hydraulic river station 540+00 to approximately 1,000 feet downstream from the SR 87 Bridge at hydraulic river station 515+00. This reach extends approximately 2,500 feet. Plate 33 shows the features of Alternative 10.1 and the 1% ACE floodplain. Plate 34 shows a detailed view of the conveyance measure for Alternative 10.1. See Section 10.2.1 for more information regarding the conveyance measure used for Alternative 10.1.

Table 14 presents the slope protection recommendations for this alternative which include riprap and soil cement/grouted stone. The slope protection recommended for this alternative is the same as Alternative 10 (See Section 10.9.3)

10.11.3 Alternative 10.2 – Levee Height based on 4% ACE

The levee heights for Alternative 10.2 are based on the water surface profiles for the 4% ACE flood. The alternative does not include additional conveyance beneath the BNSF and SR 87 Bridges as the BNSF Railroad Bridge has the capacity to convey the 4% ACE flood event without overtopping. The channel and overbank areas would remain in the baseline condition with no added excavation or widening of the channel bottom. This measure does not include removal of saltcedar. See Plate 35 for a detailed map of Alternative 10.2.

The slope protection recommended for this alternative is the same as Alternative 10 (See Section 10.9.3).

10.11.4 Alternative 10.3 – Levee Height based on 2% ACE

The levee heights for Alternative 10.3 are based on the water surface profiles for the 2% ACE flood. The alternative includes the same conveyance measure and saltcedar removal as discussed in Alternative 10.1 (See Section 10.2.1). See Plate 36 for a detailed map of Alternative 10.3.

The slope protection recommended for this alternative is the same as Alternative 10 (See Section 10.9.3).

10.11.5 Alternative 10.4 – Levee Height based on 0.5% ACE

The levee heights for Alternative 10.4 are based on the water surface profiles from the 0.5% ACE flood. The alternative includes a larger conveyance measure than Alternatives 10.1 and 10.3. It includes excavation and widening of the channel bottom from approximately 2,000 feet upstream from the BNSF Railroad Bridge at hydraulic river station 550+00 to approximately 2,400 feet downstream from the I-40 westbound bridge at hydraulic river station 480+00. This excavation and widening extends approximately 7,000 feet along the LCR. No attempt was made to deepen the channel below the existing thalweg throughout the reach. See Plate 37 & 38 for a detailed map of Alternative 10.4.

The excavation and widening of the channel begins at hydraulic river station 550+00. The width of the excavated channel varies from 200 feet at hydraulic river station 550+00 to 650 feet at the BNSF Railroad Bridge (529+39.3) to 600 feet beneath the I-40 Bridges. Downstream from the I-40 Bridges, the channel decreases to a width of approximately 150 feet at hydraulic river station 480+00. The excavation area is approximately 81 acres. This measure includes the removal of approximately 74 acres of saltcedar throughout the reach.

The slope protection recommended for this alternative is the same as Alternative 10 (See Section 10.9.3).

11.0 HYDRAULIC ANALYSIS AND RESULTS

11.1 HEC-RAS Floodplain Analysis (Assuming No Levee failure)

The 1% ACE flood was modeled for with-project alternatives to compare with the baseline condition. The six alternatives and the four optimization alternatives include measures that reduce the flood risk along the LCR, including at the Homolovi I Pueblo. The following sections comparing the with-project alternative water surface profiles to the baseline condition. Table 16 shows the comparison of WSE and flow velocity for the various alternatives compared to the baseline condition. All comparisons to the baseline condition using HEC-RAS are using Model 3 as discussed in Section 5.1. Model 3 simulates the water surface elevation on the riverside of the levee before overtopping/ probability of failure occurs. It doesn't take into account the probability of failure based on the levee fragility curve which was discussed in Section 8.3.

11.1.1 Alternative 1.1

The 1% ACE flood for Alternative 1.1 does not overtop the Winslow Levee or the RWDL. However, at the end of the rebuilt Winslow Levee, flows flank the landside of the levee before moving downstream. There are no structures within the affected area, but the floodplain boundary is very close to a few residents and outbuildings. The floodplain for the 1% ACE flood does inundate a portion of the Homolovi I Pueblo area. See Plate 27 for the 1% ACE floodplain for Alternative 1.1.

Table 16 shows the comparison of WSE and flow velocity for Alternative 1.1 compared to the baseline condition. The LCR was broken into five reaches for comparison purposes. Alternative 1.1 shows an average decrease in WSE upstream from the BNSF Railroad of 2.3 feet due to the conveyance measure. The flow velocity in this reach increased compared to the baseline condition. At the Homolovi I Pueblo, this alternative has an average decrease in WSE of 0.1 feet.

11.1.2 Alternative 3.1

The 1% ACE flood for Alternative 3.1 does not show flooding caused by overtopping or failure of the Winslow Levee. However, at the end of the rebuilt Winslow Levee, flows flank the landside of the levee before moving downstream. There are no structures within the affected area, but the floodplain boundary is very close to a few residents and outbuildings. Due to the setback of the Winslow Levee, some properties are affected by the floodplain for this alternative. There are two setback locations for this alternative. The southern setback extends from hydraulic river station 475+00 to 410+00, increasing the flow width in this reach by as much as 2,300 feet to the west. The northern setback extends from hydraulic river station 405+00 to 320+00, increasing the flow width in this reach by as much as 1,500 feet to the west. The northern setback is directly west from the Homolovi I Pueblo, but the floodplain for the 1% ACE flood does inundate a portion of the Homolovi I Pueblo area. See Plates 28 and 39 for the 1% ACE floodplain for the study area and Homolovi I Pueblo area, respectively for Alternative 3.1.

Table 16 shows the comparison of WSE and flow velocity for Alternative 3.1 compared to the baseline condition. The LCR was broken into five reaches for comparison purposes. Alternative 3.1 shows an average decrease in WSE upstream from the BNSF Railroad of 2.4 feet due to the conveyance measure. The flow velocity in this reach increased compared to the baseline condition. At the Homolovi I Pueblo, this alternative has an average decrease in WSE of 0.7 feet. See Plate 39 for the floodplain comparison.

11.1.3 Alternative 7

The floodplain for the 1% ACE flood for Alternative 7 is the same as the baseline condition as Alternative 7 does not include any structural or conveyance improvements to the Winslow Levee. See Plate 29 for the 1% ACE floodplain for Alternative 7. Alternative 7 shows no change in WSE or velocity compared to the baseline condition.

11.1.4 Alternative 8

The 1% ACE flood for Alternative 8 does not show flooding caused by overtopping or failure of the Winslow Levee. However, at the end of the rebuilt Winslow Levee, flows flank the landside of the levee before moving downstream. There do not seem to be any structures affected in this area. Also, no properties seem to be affected due to the small setback of the Winslow Levee. The setback extends from hydraulic river station 400+00 to 365+00, increasing the flow width in this reach by as much as 500 feet to the west. The setback is directly west from the Homolovi I Pueblo, but the floodplain for the 1% ACE flood does inundate a portion of the Homolovi I Pueblo area. See Plates 30 and 39 for the 1% ACE floodplain for the study area and Homolovi I Pueblo area, respectively for Alternative 8.

Table 16 shows the comparison of WSE and flow velocity for Alternative 8 compared to the baseline condition. The LCR was broken into five reaches for comparison purposes. Alternative 8 shows an average decrease in WSE upstream from the BNSF Railroad of 2.3 feet due to the conveyance measure. The flow velocity in this reach increased compared to the baseline condition. At the Homolovi I Pueblo, this alternative has an average decrease in WSE of 0.2 feet. See Plate 39 for the floodplain comparison.

11.1.5 Alternative 9

The floodplain for the 1% ACE flood for Alternative 9 is the same as the baseline condition as Alternative 9 does not include any structural or conveyance improvements to the Winslow Levee. Alternative 9 does include rebuilding the RWDL at the existing height, but the levee in this reach still becomes overtopped for the 1% ACE flood, causing flooding similar to the baseline condition. This alternative does not include conveyance improvements at the BNSF Railroad Bridge, which causes flows to attenuate at the bridge and overtop the levee upstream for the 1% ACE flood. See Plate 31 for the 1% ACE floodplain for Alternative 9. Alternative 9 shows no change in WSE or velocity compared to the baseline condition.

11.1.6 Alternative 10

The 1% ACE flood for Alternative 10 does not show flooding caused by overtopping or failure of the Winslow Levee. However, at the end of the rebuilt Winslow Levee (station 320+00 for this alternative), flows could flank the landside of the levee before moving downstream if the levee overtops/failures in the unimproved levee segment.

There are some structures affected in this area, but since this area is in the baseline conditions floodplain, flooding to this area would not be considered induced flooding when compared to the baseline condition analysis. The floodplain for the 1% ACE flood does inundate a portion of the Homolovi I Pueblo area. See Plates 32 and 39 for the 1% ACE floodplain for the study area and Homolovi I Pueblo area, respectively for Alternative 10. Alternative 10 has the same WSE and velocities as Alternative 8, including at the Homolovi I Pueblo. See Section 11.1.4 for discussion.

11.1.7 Alternative 10.1

The 1% ACE flood for Alternative 10.1 is the same as for Alternative 10. Alternative 10.1 does not include elevating residences like Alternative 10 which does not affect the floodplain. The floodplain for the 1% ACE flood does inundate a portion of the Homolovi I Pueblo area. See Plates 33 and 40 for the 1% ACE floodplain for the study area and Homolovi I Pueblo area, respectively for Alternative 10.1.

The 1% ACE flood, which has a discharge of 69,200 cubic feet per second (cfs) along the LCR near Winslow, was chosen as the design event for the eventual TSP. Under the no levee failure scenario, the water surface elevation at the Homolovi I Pueblo is decreased for Alternative 10.1 compared to the baseline condition 1% ACE flood as a result of the setback levee. The water surface elevation decreases by approximately 0.2 feet (4852.4 feet for the baseline condition 1% ACE flood and 4852.2 feet for Alternative 10.1) near the Homolovi I Pueblo. The average flow velocity in the reach near the Homolovi I Pueblo slightly decreases 0.5 fps from 4.2 fps from the baseline condition to 3.7 fps for Alternative 10.1 due to the increase in conveyance and the setback levee. See Plates 52 and 53 for a maps showing the 1% ACE floodplains near the Homolovi I Pueblo.

11.1.8 Alternative 10.2

The 4% ACE flood for Alternative 10.2 does not show flooding caused by overtopping or failure of the Winslow Levee. The floodplain for the 4% ACE flood does inundate a portion of the Homolovi I Pueblo area. See Plates 35 and 40 for the 4% ACE floodplain for the study area and Homolovi I Pueblo area, respectively for Alternative 10.2.

Table 16 shows the comparison of WSE and flow velocity for Alternative 10.2 (4% ACE flood) compared to the baseline condition (4% ACE flood). The LCR was broken into five reaches for comparison purposes. Alternative 10.2 shows no change in the WSE upstream from the BNSF Railroad. At the Homolovi I Pueblo, Alternative 10.2 shows a decrease in WSE from the baseline condition (4% ACE) of approximately 0.1 feet. See Plate 40 for the floodplain comparison at Homolovi I Pueblo.

11.1.9 Alternative 10.3

The 2% ACE flood for Alternative 10.3 does not show flooding caused by overtopping or failure of the Winslow Levee. The floodplain for the 2% ACE flood does inundate a portion of the Homolovi I Pueblo area. See Plates 36 and 40 for the 2% ACE floodplain for the study area and Homolovi I Pueblo area, respectively for Alternative 10.3.

Table 16 shows the comparison of WSE and flow velocity for Alternative 10.3 (2% ACE flood) compared to the baseline condition (2% ACE flood). The LCR was broken into five reaches for comparison purposes. Alternative 10.3 shows an average decrease in WSE upstream from the

BNSF Railroad of 3.2 feet due to the conveyance measure. The flow velocity in this reach increased compared to the baseline condition as the bridge is no longer forcing the flow to attenuate behind the bridge. At the Homolovi I Pueblo, this alternative has an average decrease in WSE of 0.1 feet. See Plate 40 for the floodplain comparison at Homolovi I Pueblo.

11.1.10 Alternative 10.4

The 0.5% ACE flood for Alternative 10.4 does not show flooding caused by overtopping or failure of the Winslow Levee. However, at the end of the rebuilt Winslow Levee (hydraulic river station 320+00 for this alternative), the river flow travels over the levee and travels upstream (south) approximately 3,000 feet. There are some structures affected in this area.

The floodplain for the 0.5% ACE flood does inundate a portion of the Homolovi I Pueblo area. See Plates 37 and 40 for the 1% ACE floodplain for the study area and Homolovi I Pueblo area, respectively for Alternative 10.

The 0.5% ACE flood, which has a discharge of 90,660 cfs along the LCR near Winslow, was chosen as the design event for the Alternative 10.4. The water surface elevation at the Homolovi I Pueblo is decreased for Alternative 10.4 compared to the baseline condition (0.5% ACE flood). The water surface elevation decreases by approximately 0.2 feet (4854 feet for the baseline condition 0.5% ACE flood and 4853.8 feet for Alternative 10.4) near the Homolovi I Pueblo.

The average flow velocity in the reach near the Homolovi I Pueblo slightly decreases approximately 0.7 fps from 4.8 fps for the baseline condition to 4.1 fps for Alternative 10.4 due to the increase in conveyance and the setback levee. See Plates 54 and 55 for a maps showing the 0.5% ACE floodplains near the Homolovi I Pueblo.

11.1.11 Comparison Summary using HEC-RAS

The baseline condition water surface elevation is consistently at or above the water surface elevations upstream from the BNSF Railroad Bridge and at the Homolovi I Pueblo for the four structural with-project alternatives (1.1, 3.1, 8, and 10). The baseline condition water surface elevation is consistently at or above the water surface elevations upstream from the BNSF Railroad Bridge and at the Homolovi I Pueblo for alternatives (10.1, 10.2, 10.3, and 10.4). Under the no levee failure scenario, the alternatives have no adverse flooding when compared with the baseline condition; including at the Homolovi I Pueblo location (see Plates 39 and 40).

Each of the structural alternatives that have conveyance improvements at the BNSF railroad Bridge show increased river velocities in that area of the river. The increased velocities could pose a potential to increase scour in the bridge area. The increase in river velocity for the alternatives compared to the baseline condition is likely due to the increased conveyance beneath BNSF Railroad Bridge which not only provides increased capacity but also alleviates backwater behind the bridge.

11.1.12 Impacts from 0.5% ACE and 0.2% ACE floods

For the structural alternatives, there are no significant changes to the 0.5% ACE and 0.2% ACE floodplains when compared to the baseline condition. Both the 0.5% and the 0.2% ACE floods result in overtopping of the BNSF Railroad Bridge which causes water to flow west along the

BNSF Railroad towards the City of Winslow. Significant flooding could occur due to the overtopping of the BNSF Railroad Bridge. For Alternative 10.4, there is no significant change for the 0.2% ACE flood when compared to the baseline condition analysis.

11.2 FLO-2D Floodplain Analysis and Comparison to Baseline Condition (Assuming Levee Failure)

11.2.1 Review of Baseline Condition FLO-2D Analyses

For the baseline condition FLO-2D simulation runs (See Section 6), five different scenarios of levee simulations were assumed. The simulation results were compared to the HEC-RAS hydraulic model results for the compatibility of the two models. The FLO-2D analyses account for volume conservation whereas the HEC-RAS hydraulic model does not.

The five FLO-2D modeling scenarios are as follows:

- Scenario 1: Assumes that LCR flows are contained by the levee.
- Scenario 2: Assumes that the levee fails totally.
- Scenario 3: Assumes that the levee fails at four different locations due to impingement and piping failures. (See Plate 21 for failure locations)
- Scenario 4: Assumes that the levee fails due to impingement near the SR 87 Bridge area. (See Plate 24 for failure locations)
- Scenario 5: Assumes that the levee fails at three locations downstream of the Bushman Acres community. (See Plate 25 for failure locations)

Scenario 1 is similar to the HEC-RAS analysis which modeled the maximum water surface elevation on the riverside of the levee before levee overtopping/failure. The Scenarios 3, 4, and 5 take into account the probability of levee failure by modeling piping and/or impingement failure locations. Scenarios 3, 4, and 5 do not have the assumption that the water surface elevations on the river-side of the levee reach the maximum level before failure occurs.

The baseline condition shows flooding in the Homolovi I Pueblo area for each of the four scenarios. The 1% ACE flow depth at Homolovi I Pueblo is the following:

- Scenario 1 (levee remains – no failure) - 5.1 feet
- Scenario 3 (4 levee failure locations) - 4.4 feet
- Scenario 4 (levee failure upstream near State Route 87 Bridge) - 4.5 feet
- Scenario 5 (levee failure downstream of Bushman Acres) - 4.9 feet.

Section 6.5 provides further discussion regarding the baseline condition FLO-2D analyses. See Table 15 for comparisons of the baseline condition hydraulic analyses.

11.2.2 Alternative 3.1

The 1% ACE flood was modeled for Alternative 3.1 using FLO-2D. See Plate 42 for the resulting floodplain showing maximum flow depths. The FLO-2D with-project analysis for the 1% ACE flood for Alternative 3.1 shows flows overtopping the Winslow Levee at the downstream end. The breakout expands across the floodplain after the levee overtopping, but major structures are not in the flow path. The maximum flow depths in this overbank area are 3 feet. At the Homolovi I Pueblo, the maximum flow depth for the 1% ACE flood is approximately 2.9 feet. The flow depth is approximately 2.2 feet less than the baseline condition which showed a flow depth of up to 5.1 feet in the Homolovi I Pueblo area. The flow velocity is approximately 3.1 fps in the Homolovi I Pueblo area for Alternative 3.1 compared to 3.4 fps for the baseline condition.

11.2.3 Alternative 10 & 10.1 Sensitivity Analysis for determination of end of levee

Alternatives 10 and 10.1 were was modeled with FLO-2D to determine the downstream location along the Winslow Levee that would be the optimal location to end improvements to the Winslow Levee. A 1% ACE floodplain was developed for five locations along the Winslow Levee. The goal was to locate where improvements would end based in the floodplain and the related economic damages. The results of the analyses showed that ending the improvements at hydraulic river station 320+00 would provide an economically viable alternative and was chosen as the downstream extent of improvements to the Winslow Levee for Alternatives 10 and 10.1. See Plate 41 for the floodplain from the FLO-2D analysis.

11.2.4 Alternatives 10 and 10.1

The FLO-2D analyses described in this section refer to Alternatives 10 and 10.1 (Referred to as Alternative 10/10.1). The 1% ACE floodplain shows flows overtopping the Winslow Levee approximately 5,000 feet downstream from the southern river impingement at hydraulic river station 320+00 (See Plate 41). The breakout overflow travels southward towards Winslow for approximately 3,000 feet. The maximum flow depth in the backwater area (upstream from hydraulic river station 320+00) is approximately 10 feet. Downstream (north) from hydraulic river station 320+00, the flows in the overbank reach a maximum flow depth of 5 feet near Ames Acres.

At the Homolovi I Pueblo, the maximum flow depth for the 1% ACE flood is approximately 4.9 feet (for Alternative 10/10.1). The flow depth decreased approximately 0.2 feet compared to the baseline conditions FLO-2D model Scenario 1 (levee remains – no failure), which showed a maximum flow depth of 5.1 feet in the Homolovi I Pueblo area. The decrease in WSE of 0.2 feet equates to an average decrease (Alternative 10/10.1 vs Scenario 1) in flooded area of 2 feet along the edge of the floodplain at Homolovi I Pueblo. The flow velocity is approximately 3.2 fps in the Homolovi I Pueblo area for Alternative 10/10.1 compared to 3.4 fps for the baseline condition. See Plate 56 for the floodplain comparison at Homolovi I Pueblo. See Table 15.

For Scenarios 3, 4, and 5, which account for the probability of levee failure, the WSE for Alternative 10/10.1 was at or above the baseline condition WSE for the 1% ACE flood. The maximum flow depths for Scenarios 3, 4, and 5 are approximately 4.4, 4.5, and 4.9 feet, respectively. Compared to the baseline condition analyses for Scenarios 3, 4, and 5, the WSE for Alternative 10/10.1 increases by approximately 0.5, 0.4 and 0 feet, respectively. The increase in WSE of 0.5 feet (Alternative 10/10.1 vs Scenario 3 – levee failure) equates to an average increase in flooded area of 10 feet along the edge of the floodplain at Homolovi I Pueblo. The flow velocity near Homolovi I Pueblo is approximately 3.1 fps for Scenario 3, 3.2 fps for Scenario 4, and 3.4 fps for Scenario 5. Alternatives 10 and 10.1 comparatively have an approximate velocity near Homolovi I Pueblo of 3.2 fps under Scenario 1. See Table 15.

11.2.5 Other Alternatives

Some with-project alternatives were not modeled individually in FLO-2D; however, the results would be similar to other analyses as following:

- Alternatives 1.1 and 8 would be similar to baseline condition Scenario 1 – no levee failure due to the levee improvements removing the probability of unsatisfactory performance of the Winslow Levee. Refer to Table 15 for the floodplain characteristics for the baseline condition scenario 1.
- Alternative 7 would be similar to baseline condition – Scenario 3 – levee failure at four locations due to the probability of unsatisfactory performance of the Existing Winslow Levee near the BNSF Railroad Bridge and downstream near the impingement locations. Refer to Table 15 for the floodplain characteristics for the baseline condition scenario 3.
- Alternative 9 would be similar to baseline condition Scenario 5 – levee failure downstream near the impingement and piping locations. Refer to Table 15 for the floodplain characteristics for the baseline condition scenario 5.
- Alternative 10.2 (4% ACE) would have similar floodplain comparison as Alternative 10.1 and the baseline condition levee failure Scenario 4 due to the probability of unsatisfactory performance of the Existing Winslow Levee near the BNSF Railroad Bridge for the 4% ACE flood event. Refer to Table 15 for the floodplain characteristics for Alternative 10.1.
- Alternative 10.3 (2% ACE) would have a similar floodplain comparison as Alternative 10.1 and the baseline condition levee failure Scenario 4 due to the probability of unsatisfactory performance of the Existing Winslow Levee near the BNSF Railroad Bridge for the 2% ACE flood event. Refer to Table 15 for the floodplain characteristics for Alternative 10.1.
- Alternative 10.4 (0.5% ACE) would have a similar floodplain comparison as Alternative 10.1 and the baseline condition levee failure Scenario 3 due to the probability of unsatisfactory performance of the Existing Winslow Levee near the BNSF Railroad Bridge and near the impingement locations downstream from Homolovi I Pueblo. Refer to Table 15 for the floodplain characteristics for Alternative 10.1.

See Table 15 for maximum flow depth and comparisons to Alternative 10.1. Table 16 can be referenced for flow depth comparison for each alternative to the baseline condition for the area around the bridges. The flow depth comparisons shown in Table 16 were determined using HEC-RAS.

11.2.6 Potential Flooding Impacts at Homolovi I Pueblo

Compared to the levee failure baseline condition scenarios (3, 4, and 5), Alternative 10 and 10.1 would result in an increase in WSE at Homolovi I Pueblo by up to 0.5 feet and increase the flow velocity by up to 0.2 fps. Compared to the no levee failure baseline condition scenario (1), Alternatives 10 and 10.1 would result a decrease is WSE at Homolovi I Pueblo by 0.2 feet. Under Scenarios 3, 4, and 5, the improved levee in Alternatives 10 and 10.1 would result in a minor increase in flooding. The difference in WSE between the levee not failing (Scenario 1) and the levee failing (Scenarios 3-5) is a maximum of 0.7 feet for the baseline condition.

11.3 Comparison of With-Project Alternatives and Baseline Condition Using Conditional Non-Exceedance Probabilities

Engineer Circular 1110-2-6067 (Reference J) was used to determine the required levee height above the water surface (freeboard). Section 3.1.3 provides an overview for the Conditional Non-Exceedance Probability (CNP) and the levee freeboard. To meet the National Flood Insurance Program (NFIP) levee system evaluation requirements, a levee must have at least 90% assurance of excluding the 1% ACE flood for all reaches of the system. For levees, if the top of levee elevation is less than the FEMA required freeboard above the 1% ACE flood stage, then the levee can only be in accordance with NFIP levee system evaluation requirements if the assurance (CNP) is 95% or greater. The top of levee elevation shall not be less than two feet above the 1% ACE flood elevation, even if assurance is 95% or greater.

Table 17 shows the project performance and the Conditional Non-Exceedance Probabilities for the baseline condition and the with-project alternatives. These statistics are broken down into the median and expected annual exceedance probabilities, the long term risk, and the conditional non-exceedance probabilities by event. The target annual exceedance probability is broken into a median and expected probability that the levee will be over topped. The long term risk is the probability that a target stage will be exceeded in a 10, 30 and 50 year period. The Economics Appendix provides more information and discussion regarding these terms.

The CNP for the 1% ACE will be discussed in the following sections. Index Reaches 1 and 2 (shown on Plate 26) are used in the economic analysis for the damage assessment and project performance calculations. Index Reach 1 covers the Winslow Area and the upstream portion of the Winslow Levee to hydraulic river station 350+00. Index Reach 2 begins and hydraulic river station 350+00 and extends to the end of the existing Winslow Levee at hydraulic river station 170+00.

The CNP for the baseline condition for the 1% ACE flood is 0.072 for Index Reach 1, meaning that the existing Winslow Levee has a 7.2% assurance or chance of excluding the 1% ACE flood. The CNP for the baseline condition is 0.506 for Index Reach 2.

11.3.1 Alternative 1.1

The CNP for Alternative 1.1 for the 1% ACE flood is 0.880 for Index Reach 1, meaning that the existing Winslow Levee has an 88% assurance or chance of excluding the 1% ACE flood. The CNP for Index Reach 2 is 0.948.

11.3.2 Alternative 3.1

The CNP for Alternative 3.1 for the 1% ACE flood is 0.873 for Index Reach 1, meaning that the existing Winslow Levee has an 87.3% assurance or chance of excluding the 1% ACE flood. The CNP for Index Reach 2 is 0.948.

11.3.3 Alternative 7

The CNP for Alternative 7 is the same as the baseline condition described in Section 11.3.

11.3.4 Alternative 8

The CNP for Alternative 8 for the 1% ACE flood is 0.880 for Index Reach 1, meaning that the existing Winslow Levee has an 88% assurance or chance of excluding the 1% ACE flood. The CNP for Index Reach 2 is 0.948.

11.3.5 Alternative 9

The CNP for Alternative 9 is the same as the baseline condition described in Section 11.3.

11.3.6 Alternative 10

The CNP for Alternative 10 for the 1% ACE flood is 0.880 for Index Reach 1, meaning that the existing Winslow Levee has an 88% assurance or chance of excluding the 1% ACE flood. The CNP for Index Reach 2 is 0.505.

11.3.7 Alternative 10.1

The CNP for Alternative 10.1 for the 1% ACE flood is 0.880 for Index Reach 1, meaning that the existing Winslow Levee has an 88% assurance or chance of excluding the 1% ACE flood. The CNP for Index Reach 2 is 0.505. Alternative 10.1 levee improvements end at hydraulic river station 320+00, which is in Index Reach 2. It is required that all levee reaches have an assurance of 90%. For this alternative, it is assumed that Index Reach 1 would extend to hydraulic river station 320+00 or the end of levee improvements. Alternative 10.1 was formulated assuming that it would provide at least 90% assurance of containing the 1% ACE event. However, as shown on these results on Table 17, it only provides an 88% assurance level. Additional analysis was conducted indicating that 0.3 feet of additional height will increase the assurance to a 90% level for the 1% ACE event, resulting in a levee height that is 3.3 feet above the water surface profile.

11.3.8 Alternative 10.2

The CNP for Alternative 10.2 for the 1% ACE flood is 0.352 for Index Reach 1, meaning that the existing Winslow Levee has a 35.2% assurance or chance of excluding the 1% ACE flood. The CNP for Index Reach 2 is 0.505. Index Reach 1 extends to the end of levee improvements (hydraulic river station 320+00).

11.3.9 Alternative 10.3

The CNP for Alternative 10.3 for the 1% ACE flood is 0.694 for Index Reach 1, meaning that the existing Winslow Levee has a 69.4% assurance or chance of excluding the 1% ACE flood. The CNP for Index Reach 2 is 0.506. Index Reach 1 extends to the end of levee improvements (hydraulic river station 320+00).

11.3.10 Alternative 10.4

The CNP for Alternative 10.4 for the 0.5% ACE flood is 0.962 for Index Reach 1, meaning that the existing Winslow Levee has a 96.2% assurance or chance of excluding the 0.5% ACE flood. The CNP for Index Reach 2 is 0.507. Index Reach 1 extends to the end of levee improvements (hydraulic river station 320+00).

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12.0 HYDRAULIC ANALYSIS AT HOMOLOVI I PUEBLO

12.1 Homolovi I Pueblo Overview

The Homolovi I Pueblo is an archeological site on the river side of the Winslow Levee that is in the 1% ACE floodplain. This section includes discussion of measures considered to alleviate the flooding concern at the Homolovi I Pueblo. Section 5.1 discusses the baseline condition at the Homolovi I Pueblo. See Plate 43 for the baseline condition 1% ACE floodplain at the Homolovi I Pueblo compared to the 1993 flood event floodplain (Reference K). Attachment 6 provides further information regarding the Homolovi Pueblo Hydraulic Analysis.

12.2 Homolovi I Pueblo Flood Risk Reduction Measures

Seven flood risk reduction measures (FRRM) were considered to reduce the flooding impact at the Homolovi I Pueblo. The alternatives include one or a combination of the following: removing the existing levee downstream from hydraulic river station 320+00, removing saltcedar downstream from the Homolovi I Pueblo, setting back the Winslow Levee, channelization downstream from the Homolovi I Pueblo, and a storage area to capture the peak runoff.

12.2.1 Alternative 10 with Removal of the Winslow Levee downstream from Station 320+00

This alternative (FRRM 1) is based on Alternative 10 (See Section 10.9) but has a key difference. This measure includes the removal of the Winslow Levee downstream of hydraulic river station 320+00, which could potentially reduce the water surface elevation (WSE) at the Homolovi I Pueblo. See Plate 44.

12.2.2 Alternative 3.1 with Removal of the Winslow Levee downstream from Station 320+00

This alternative (FRRM 2) is based on Alternative 3.1 (See Section 10.5). It differs in that this measure includes the removal of the Winslow Levee downstream from hydraulic river station 320+00, which could potentially reduce the WSE at the Homolovi I Pueblo See Plate 45.

12.2.3 Alternatives 8 and 10 with Saltcedar Removal

This alternative (FRRM 3) is based on Alternatives 8 and 10, but the analysis includes the removal of saltcedar downstream from the Homolovi I Pueblo along the western bank of the LCR. The removal of saltcedar is an attempt to decrease the Manning's n-value from 0.12 (dense saltcedar) to 0.05 (floodplains, scattered brush, heavy weeds) in order to decrease the WSE near the Homolovi I Pueblo. Removal of saltcedar in this area would further decrease the flooding impacts at Homolovi I Pueblo. See Plate 46.

12.2.4 Alternative 3.1 with Saltcedar Removal

This alternative (FRRM 4) is based on Alternative 3.1 (See Section 10.5), but includes the removal of saltcedar downstream from the Homolovi I Pueblo along the western bank of the LCR. The removal of saltcedar is an attempt to decrease the Manning's n-value in order to decrease the WSE near the Homolovi I Pueblo. See Plate 47.

12.2.5 Alternative 3.1 with Saltcedar Removal and Winslow Levee removal downstream from Station 320+00

This alternative (FRRM 5) is based on Alternative 3.1 (See Section 10.5), but includes the removal of saltcedar downstream from the Homolovi I Pueblo along the western bank of the LCR and the removal of the Winslow Levee downstream from hydraulic river station 320+00. The removal of saltcedar and the removal of the downstream section of the levee is an attempt to decrease the Manning's n-value in order to decrease the WSE near the Homolovi I Pueblo. See Plate 48.

12.2.6 Alternative 3.1 with Channelization of the LCR near Homolovi I Pueblo

This alternative (FRRM 6) is based on Alternative 3.1 (See Section 10.5), but includes channelization of the LCR from hydraulic river station 390+00 to 325+00. The channelization is another attempt to decrease the WSE at the Homolovi I Pueblo. See Plate 49.

12.2.7 Upstream Storage Area / Detention Basin

This alternative (FRRM 7) consists of creating a storage basin large enough to capture and hold the difference between the 1% ACE and the 2% ACE peak flood hydrograph. The flood hydrograph for the 86 hour event begins to peak at 32 hours and lasts to 36 hours for both flood events. The difference in the peak flood volume between the two flood events is approximately 3,900 acre-feet, which corresponds to a 390 acre basin that is 10 feet deep. This alternative could potentially store the difference in peak flow between the 1% ACE and 2% ACE flood peak hydrograph, which could decrease the WSE at Homolovi I Pueblo.

12.3 Comparison of Winslow Alternatives to Baseline Condition at Homolovi I Pueblo

Section 11.1 provides a comparison of the Winslow Alternatives to the Baseline Condition at the Homolovi I Pueblo.

12.4 Comparison of Flood Risk Reduction Measures to Baseline Condition at Homolovi I Pueblo

This section compares the WSEs for the six flood risk reduction measures at Homolovi I Pueblo to the baseline condition assuming no levee failure. The flood risk reduction measures each provide a lower WSE at the Homolovi I Pueblo when looking at hydraulic river station 390+00. The differences in WSE among the flood risk reduction measures at Homolovi I Pueblo and the baseline condition vary from approximately 0.6 to 2.5 feet. The drop in WSE for the six flood risk reduction measures compared to the baseline condition assuming no levee failure is:

- 0.7 feet for FRRM 1 (Section 12.2.1)
- 1.3 feet for FRRM 2 (Section 12.2.2)
- 1.3 feet for FRRM 3 (Section 12.2.3)
- 2.1 feet for FRRM 4 (Section 12.2.4)
- 2.5 feet for FRRM 5 (Section 12.2.5)
- 1.7 feet for FRRM 6 (Section 12.2.6)

FRRM 5, the flood risk reduction measure that includes saltcedar removal and removal downstream of hydraulic river station 320+00, provides the lowest WSE among the six flood risk reduction measures at Homolovi I Pueblo.

The flood risk reduction measure with the storage area was not modeled in HEC-RAS due to the real estate concerns and the need for nearly 400 acres of land needed to store the peak runoff (assuming a basin of approximately 10 feet deep).

See Table 18 for the comparison of WSE for the six flood risk reduction measures at Homolovi I Pueblo to the Baseline Condition. See Plate 50 for the floodplains for the flood risk reduction measures at Homolovi I Pueblo.

12.5 Homolovi I Pueblo Conclusion/Recommendation

Seven flood risk reduction measures discussed in the sections above are designed to reduce the flooding impact at the Homolovi I Pueblo. The measures include one or a combination of the following: removing the existing levee at hydraulic river station 320+00, removing saltcedar downstream from the Homolovi I Pueblo, setting back the Winslow Levee, channelization downstream from the Homolovi I Pueblo, and a storage area to capture the peak runoff. The flood risk reduction measures at Homolovi I Pueblo considered in this analysis decrease the water surface elevation compared to the baseline condition. Alternatively, to mitigate the flooding impacts at the Homolovi I Pueblo, a ring levee could be used to protect the archeological site.

None of the flood risk reduction measures have been incorporated into the TSP.

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13.0 SUMMARY AND CONCLUSIONS

13.1 Interior Drainage

An interior drainage analysis was completed for the baseline condition. Two existing culverts convey flows from the land side of the levee to the river side. The K-3 Channel conveys flows to three concrete box culverts (10 feet wide by 4 feet high) which travel beneath I-40. Plate 51 shows the interior drainage analysis within the project area. The K-3 hydraulic gates receive flow from a drainage area of approximately 510 acres. The I-4 Channel conveys flows to four concrete box culverts (10 feet wide by 4 feet high) which travel beneath the existing Winslow Levee. The (I-4) hydraulic gates receive flow from a drainage area of approximately 695 acres. There are three current storage areas on the land side of the levee. See Table 19 for pertinent data for the interior drainage features.

13.2 Baseline Condition Analysis

13.2.1 Overview

Baseline condition channel hydraulic and floodplain studies were conducted for the Little Colorado River near Winslow (Sections 5.0 and 6.0). A sedimentation analysis was also conducted for the LCR Winslow reach (Section 7.0). The studies included hydraulic and sediment data analyses, a site visit, development of HEC-RAS models (including sediment transport), development of a FLO-2D model, and simulation and delineation of floodplain.

13.2.2 Hydraulic Model Development

HEC-RAS models were developed for the baseline condition for the reach of the LCR near Winslow. Geotechnical and hydraulic analyses by the USACE show that the Winslow Levee could safely convey up to 2.7% ACE flood discharge. Steady state HEC-RAS model simulations were applied to the Winslow study reach. Overflow maps for the Winslow Levee study area from 50% to 0.2% ACE floods were produced from the steady state HEC-RAS simulations.

Baseline condition Floodplain analysis shows that the Ruby Wash Diversion Levee and Winslow Levee near the Ruby Wash confluence with the LCR do not offer sufficient protection for the 2%, 1%, 0.5%, and 0.2% ACE floods. For the 2% ACE flood the RWDL overtops along the left bank near the confluence with the LCR (hydraulic river station 535+00). The flooding is contained between the RWDL and the BNSF Railroad. The amount of flow through the BNSF Railroad underpasses is negligible considering the duration of the overflow and the volume that is stored south of the railroad embankment. The 1%, 0.5%, and 0.2% ACE floods also overtop the RWDL, resulting in flooding in the left overbank from approximately hydraulic river station 542+50 to the confluence with the LCR. When combined with the overtopping of the Winslow Levee between the BNSF Railroad Bridge and the SR 87 Bridge, the floodplain shows extensive flooding in the City of Winslow.

The steady state HEC-RAS model simulations assume that there is unlimited water supply to the system. In reality, the overflow process is a dynamic flow limited by volume of water supply. In order to further check the overflow maps produced by HEC-RAS steady state simulations, a two-dimensional flow volume conservation model was applied to the Winslow study area. Five FLO-2D models were developed for the floodplain analysis and are detailed in Section 6.0.

13.2.3 Comparison of Hydraulic Models

Comparisons of the HEC-RAS model results with the FLO-2D model results show that the results are compatible and in agreement. In addition, as demonstrated in the FLO-2D simulation scenarios, a levee breach failure in the upstream area (Plate 24) would cause significant damage to the city of Winslow, Ames Acres, Bushman Acres, and other areas behind the levee. The levee breach failure in the downstream area near Bushman Acres (Plate 25) does not cause damages to the city of Winslow, Ames Acres, Bushman Acres, and other business areas behind the levee.

HEC-RAS model results were used for the economic flood damage analysis. The HEC-RAS model was also used to produce a Risk & Uncertainty Analysis. FLO-2D model results were used to complement HEC-RAS model results as well as to provide input for plan formulation. Both the HEC-RAS and FLO-2D models will be used to evaluate project alternatives in the next phase of the study.

13.2.4 Sediment Transport Analysis

A baseline condition sediment transport model was created using the geometry file from the baseline conditions hydraulic model which included approximately 10 miles of stream northeast from Winslow (most of which was outside of the study area). No model calibration was conducted for the sediment analysis. The sediment analysis was based on best available data which included data from the USBR study in 2003 (Reference E) and the 2009 Winslow survey data (Reference C) completed by Navajo County, as well as technical manuals.

13.2.5 Conditional Non-Exceedance Probability

The CNP for the baseline condition for the 1% ACE flood is 0.072 for Index Reach 1, meaning that the existing Winslow Levee has a 7.2% assurance or chance of excluding the 1% ACE flood. The CNP for the baseline condition is 0.506 for Index Reach 2.

13.3 With-Project Alternatives Analysis

13.3.1 Overview

The with-project analysis began with conveyance measure analysis for flow under the BNSF Railroad Bridge. See Attachment 5 for further information. The selected conveyance measure was Conveyance Measure C which includes excavation and widening of the channel bottom beginning approximately 1,000 feet upstream from the BNSF Railroad and extending approximately 1,000 feet downstream from the SR 87 Bridge. This measure includes a natural channel bottom and does not include concrete lining.

13.3.2 Floodplain Analysis

The floodplains for the 1% ACE flood for Alternative 1.1 and Alternative 8 do not show flooding caused by failure of the Winslow Levee. However, at the end of the rebuilt Winslow Levee, flows flank the landside of the levee before moving downstream. There are no structures within the affected area, but the floodplain boundary is very close to a few residents and outbuildings. However, a cutoff levee could potentially minimize the flooding in the overbank area upstream from the end of improvements. The location of the end of improvements was selected based on a sensitivity analysis on the flood inundation area near the end of improvements. Due to the terrain on the land side of the levee, ending the improvements 1,500 feet upstream would only move the flooding problem further upstream. The current location of the end of improvements was chosen to minimize flooding in this area. The floodplain for the 1% ACE flood does inundate the Homolovi I Pueblo area (Plates 27 and 30).

The floodplain for the 1% ACE flood for Alternative 3.1 does not show flooding caused by failure of the Winslow Levee. However, at the end of the Rebuilt Winslow Levee, flows travel around the end of the levee and travel upstream (south) approximately 1,500 feet. There are no structures within the affected area, but the floodplain boundary is very close to a few residents and outbuildings. There do not seem to be any structures affected in this area. Due to the setback of the Winslow Levee, some properties are affected by the floodplain for this alternative. The northern setback is directly west from the Homolovi I Pueblo. The floodplain for the 1% ACE flood does inundate the Homolovi I Pueblo area (Plate 28). The FLO-2D with-project analysis for the 1% ACE flood for Alternative 3.1 shows flows overtopping the Winslow Levee at the downstream end. The breakout expands across the floodplain after the levee overtopping, but major structures do not appear to be in the flow path. The maximum flow depth in this overbank area is 3 feet.

The floodplains for the 1% ACE flood for Alternative 7 and Alternative 9 are the same as the baseline condition as the floodplains do not include any conveyance improvements to the Winslow Levee. See Plates 29 and 31.

The floodplain for the 1% ACE flood for Alternative 10 does not show flooding caused by failure of the Winslow Levee. However, at the end of the rebuilt Winslow Levee (hydraulic river station 320+00 for this alternative), the overbank river flow travels around and over the levee and travels upstream (south) approximately 3,000 feet. This assumes that the unimproved levee segment downstream from the federal project fails. There are some structures affected in this area, but impact to these properties would not change compared to the existing condition. The floodplain for the 1% ACE flood does inundate the Homolovi I Pueblo area.

Alternative 10 was modeled using FLO-2D for the 1% ACE flood. The floodplain shows overtopping of the Winslow Levee near hydraulic river station 320+00. The breakout overflow travels southward towards Winslow for approximately 3,000 feet. The maximum flow depth in this area upstream from 320+00 is approximately 10 feet. The WWTP is protected by a ring levee with 1% ACE flood design; however, the area around the WWTP is inundated with a maximum flow depth of 3 feet. Downstream (north) from hydraulic river station 320+00, the flows in the overbank reach a maximum flow depth of 5 feet near Ames Acres.

Detailed analysis of each alternative is presented in Section 11. Conclusions regarding the TSP and the alternative with the greatest potential for increase in water surface elevation are discussed in the following sections.

13.3.3 Floodplain Comparison Alternative 10.1

A portion of the Homolovi I Pueblo footprint is currently within the 1% ACE floodplain for the baseline condition based on hydraulic analysis comparisons discussed in Sections 11.2 and 11.3 for the FLO-2D and HEC-RAS models, respectively.

The comparison of the HEC-RAS baseline condition (which assumes maximum WSE before levee failure on the riverside of the levee) and with-project alternatives show that the baseline condition water surface elevation is consistently at or above the water surface elevations of the with-project alternatives under Scenario 1 (no levee failure). The alternatives have no adverse effect when compared with the baseline condition; including at the Homolovi I Pueblo (see Plates 39 and 40). Alternatives 10 and 10.1 result in a decreased floodplain extent and a decreased water surface elevation at Homolovi I Pueblo compared to the baseline condition. Alternatives 10 and 10.1 have the same flood duration as the baseline condition. See Plates 52 and 53 for maps comparing the baseline condition 1% ACE floodplain to the TSP floodplain at the Homolovi I Pueblo.

Based on the FLO-2D analysis, compared to the levee failure baseline condition scenarios (3, 4, and 5), Alternative 10 and 10.1 would result in an increase in WSE at Homolovi I Pueblo by up to 0.5 feet and increase the flow velocity by up to 0.2 fps. Compared to the no levee failure baseline condition scenario (1), Alternatives 10 and 10.1 would result a decrease is WSE at Homolovi I Pueblo by 0.2 feet. The improved levee in Alternatives 10 and 10.1 would result in a minor increase in flooding. The difference in WSE between the levee not failing (Scenario 1) and the levee failing (Scenarios 3-5) is a maximum of 0.7 feet for the baseline condition.

13.3.4 Floodplain Comparison Alternative 10.4

Hydraulic analysis shows that Alternative 10.4 would slightly decrease flooding at the Homolovi I Pueblo compared to the baseline condition 0.5% ACE flood under the no levee failure scenario (Scenario 1). A portion of the Homolovi I Pueblo footprint is currently within the 0.5% ACE floodplain for the baseline condition. Alternative 10.4 results in a decreased floodplain extent at Homolovi I Pueblo compared to the baseline condition based on the HEC-RAS hydraulic analysis. Alternative 10.4 has the same flood duration as the baseline condition. Implementation of Alternative 10.4 would decrease the footprint of the floodplain at the Homolovi I Pueblo. See Plates 54 and 55 for maps comparing the baseline condition 0.5% ACE floodplain to Alternative 10.4 0.5% ACE floodplain at the Homolovi I Pueblo.

Alternative 10.4 has a small area near Interstate 40 and Route 66 east of the LCR that has induced flooding as a result of Alternative 10.4 measures compared to the baseline condition 0.5% ACE floodplain. See Plate 54. Hydraulic analysis shows that Alternative 10.4 would slightly decrease flooding at the Homolovi I Pueblo compared to the baseline condition 0.5% ACE flood under the no levee failure scenario. See Plate 55.

13.3.5 Conditional Non-Exceedance Probability

The CNP for Alternative 10.1 for the 1% ACE flood is 0.880 for Index Reach 1, meaning that the existing Winslow Levee has an 88% assurance or chance of excluding the 1% ACE flood. Additional analysis was conducted indicating that 0.3 feet of additional height will increase the assurance to a 90% level for the 1% ACE event, resulting in a levee height that is 3.3 feet above the water surface profile.

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TABLES

LITTLE COLORADO RIVER AT WINSLOW
HYDRAULIC AND SEDIMENTATION APPENDIX
APRIL 2016

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Table 1: Discharge-Frequency Values for the Steady State HEC-RAS Model

Frequency		Discharge
% ACE	**(year)**	**(cfs)**
50	2	8,070
20	5	16,360
10	10	24,400
4	25	38,310
2	50	52,020
1	100	69,200
0.5	200	90,660
0.2	500	127,250
ACE = Annual Chance Exceedance		

Table 2: Peak and Volume Discharge Frequency Values for LCR at Winslow

	Volume (cfs) per Frequency Event (%ACE)							
	50	**20**	**10**	**4**	**2**	**1**	**0.5**	**0.2**
Peak Q	8,070	16,360	24,400	38,310	52,020	69,200	90,660	127,250
1-Day Volume	4,030	9,480	13,810	19,630	24,020	28,370	32,620	38,080
2-Day Volume	3,320	7,800	11,350	16,120	19,730	23,290	26,780	31,260
3.5-day Volume	2,560	5,875	8460	11,880	14,440	16,945	19,390	22,510

Table 3: Roughness Coefficients (Manning's n-values)

Manning's n-value	Description
0.03	Channel Bed with fine to medium sand[1]
0.05	Floodplains, scattered brush, heavy weeds[2]
0.085	Floodplains, Medium to dense brush[2]
0.09	Residential Medium Density[3]
0.12	Dense willow, heavy timber, "saltcedar"[2]
Sources	
1	*Design Manual for Engineering Analysis of Fluvial Systems, ADWR, 1985*
2	*Open Channel Hydraulics, Chow, 1959*
3	*A method for adjusting values of Manning's roughness coefficients for flooded urban areas, Hejl, 1977*

Table 4: Bridges in Study Area

Crossing Name	HEC-RAS Station	# Piers	Span Length (feet)	Pier Shape	Pier Width (feet)	Bridge Width (feet)
BNSF Railroad Bridge	529+39.27	5	700	Triangular Nose 120 degree Angle	7.0 to 8.5	36
State Route 87 / Route 66	524+13.68	6	850	Circular	6	45
Interstate 40 Eastbound	505+88.40	12	1030	Elongated piers with semi-circular ends	2	40
Interstate 40 Westbound	504+71.60	12	1030	Elongated piers with semi-circular ends	2	40
Notes:						
The Pier Width for the BNSF Railroad Bridge is 8.5 near the ground and 7.0 near the bridge deck						
See Plate 3 for Bridge Locations						

Table 5: Calculated Inundated Area for Total Levee Failure

		Discharge Frequency (%ACE)		
		1	**0.5**	**0.2**
Total Volume	**(acre-feet)**	122,502	140,114	161,962
Inundated Area	**(acres)**	12,411	12,775	13,245
Note: The duration of the hydrograph was 3.5 days				

Table 6: Calculated Inundated Area for Impingement and Piping Failure

		Discharge Frequency (%ACE)		
		1	**0.5**	**0.2**
Total Volume	**(acre-feet)**	122,502	140,114	161,962
Inundated Area	**(acres)**	12,564	12,999	13,489
Note: The duration of the hydrograph was 3.5 days				

Table 7: Downstream Boundary Rating Curve (HEC-RAS Sediment Model)

Frequency (% ACE)	**Discharge (cfs)**	**W.S. Elev. (feet)**
Low Flow	500	4814.3
50	8,070	4817.3
20	16,360	4818.1
10	24,400	4818.7
4	38,310	4819.3
2	52,020	4819.9
1	69,200	4820.5
0.5	90,660	4821.1
0.2	127,250	4822.0

Table 8: Expected & Computed Peak Discharges with Confidence Limits for LCR at Winslow

Exceedance Probability (%)	**Return Period (Year)**	**95% Confidence Limit (cfs)**	**Computed Peak Discharge (cfs)**	**Expected Peak Discharge (cfs)**	**5% Confidence Limit (cfs)**
0.2	500	81,120	127,250	158,760	240,380
0.5	200	60,680	90,660	106,440	159,000
1	100	48,090	69,200	78,060	114,590
2	50	37,540	52,020	56,740	81,240
4	25	28,720	38,310	40,630	56,360
10	10	19,220	24,400	25,160	33,140
20	5	13,320	16,360	16,610	20,960
50	2	6,670	8,070	8,070	9,760

Table 9: Bed Identifier Characteristics

Material	**Identifier**
Rock/Resistant Clay	0
Boulders	1
Cobbles	2
Gravels	3
Sands	4
Engineering Manual EM 1110-2-1619, Risk-Based Analysis (Table 5-1)	
For Flood Damage Reduction Studies, 1 August 1996	

Table 10 - Risk and Uncertainty

			Existing Baseline Condition			Alternative 1.1		
Reach & Index Cross-Section	Annual Chance Exceedance (%)	Q (cfs)	Natural Uncertainty ($S_{natural}$)	Model Uncertainty (S_{model})	Total Uncertainty (S_{Total})	Natural Uncertainty ($S_{natural}$)	Model Uncertainty (S_{model})	Total Uncertainty (S_{Total})
Index Reach 1	0.2	127,250			0.95			1.00
Upstream Sta	0.5	90,660			0.95			1.00
545+00	1	69,200	0.54	0.78	0.95	0.56	0.83	1.00
Downstream Sta	2	52,020			0.71			0.75
350+00	4	38,310			0.53			0.55
Index Sta.	10	24,400			0.33			0.35
535+00	20	16,360			0.30			0.30
	50	8,070			0.30			0.30
Index Reach 2	0.2	127,250			1.09			1.09
Upstream Sta	0.5	90,660			1.09			1.09
350+00	1	69,200	0.54	0.95	1.09	0.55	0.95	1.09
Downstream Sta	2	52,020			0.82			0.82
170+00	4	38,310			0.60			0.60
Index Sta.	10	24,400			0.38			0.38
290+00	20	16,360			0.30			0.30
	50	8,070			0.30			0.30
			Alternative 3.1			Alternative 7 & 9		
Reach & Index Cross-Section	Annual Chance Exceedance (%)	Q (cfs)	Natural Uncertainty ($S_{natural}$)	Model Uncertainty (S_{model})	Total Uncertainty (S_{Total})	Natural Uncertainty ($S_{natural}$)	Model Uncertainty (S_{model})	Total Uncertainty (S_{Total})
Index Reach 1	0.2	127,250			0.99			0.95
Upstream Sta	0.5	90,660			0.99			0.95
545+00	1	69,200	0.56	0.82	0.99	0.54	0.78	0.95
Downstream Sta	2	52,020			0.74			0.71
350+00	4	38,310			0.55			0.53
Index Sta.	10	24,400			0.35			0.33
535+00	20	16,360			0.30			0.30
	50	8,070			0.30			0.30
Index Reach 2	0.2	127,250			1.09			1.09
Upstream Sta	0.5	90,660			1.09			1.09
350+00	1	69,200	0.55	0.95	1.09	0.54	0.95	1.09
Downstream Sta	2	52,020			0.82			0.82
170+00	4	38,310			0.60			0.60
Index Sta.	10	24,400			0.38			0.38
290+00	20	16,360			0.30			0.30
	50	8,070			0.30			0.30

Table 10 - Risk and Uncertainty (continued)

			Alternatives 8 & 10 & 10.1			Alternative 10.2		
Reach & Index Cross-Section	Annual Chance Exceedance (%)	Q (cfs)	Natural Uncertainty ($S_{natural}$)	Model Uncertainty (S_{model})	Total Uncertainty (S_{Total})	Natural Uncertainty ($S_{natural}$)	Model Uncertainty (S_{model})	Total Uncertainty (S_{Total})
Index Reach 1	0.2	127,250			0.99			0.77
Upstream Sta	0.5	90,660			0.99			0.77
545+00	1	69,200	0.56	0.82	0.99	0.52	0.57	0.77
Downstream Sta	2	52,020			0.74			0.58
350+00	4	38,310			0.55			0.43
Index Sta.	10	24,400			0.35			0.27
535+00	20	16,360			0.30			0.30
	50	8,070			0.30			0.30
Index Reach 2	0.2	127,250			1.07			1.03
Upstream Sta	0.5	90,660			1.07			1.03
350+00	1	69,200	0.54	0.92	1.07	0.53	0.88	1.03
Downstream Sta	2	52,020			0.80			0.77
170+00	4	38,310			0.59			0.57
Index Sta.	10	24,400			0.38			0.36
290+00	20	16,360			0.30			0.30
	50	8,070			0.30			0.30

			Alternative 10.3			Alternative 10.4		
Reach & Index Cross-Section	Annual Chance Exceedance (%)	Q (cfs)	Natural Uncertainty ($S_{natural}$)	Model Uncertainty (S_{model})	Total Uncertainty (S_{Total})	Natural Uncertainty ($S_{natural}$)	Model Uncertainty (S_{model})	Total Uncertainty (S_{Total})
Index Reach 1	0.2	127,250			0.96			1.13
Upstream Sta	0.5	90,660			0.96			1.13
545+00	1	69,200	0.53	0.79	0.96	0.57	0.98	1.13
Downstream Sta	2	52,020			0.74			0.58
350+00	4	38,310			0.55			0.43
Index Sta.	10	24,400			0.35			0.27
535+00	20	16,360			0.30			0.30
	50	8,070			0.30			0.30
Index Reach 2	0.2	127,250			1.06			1.07
Upstream Sta	0.5	90,660			1.06			1.07
350+00	1	69,200	0.53	0.91	1.06	0.55	0.92	1.07
Downstream Sta	2	52,020			0.80			0.77
170+00	4	38,310			0.59			0.57
Index Sta.	10	24,400			0.38			0.36
290+00	20	16,360			0.30			0.30
	50	8,070			0.30			0.30

Table 11: Combined Probability of Unsatisfactory Performance (Levee Fragility)

Ruby Wash Diversion Levee/Winslow Levee			**Winslow Levee**		
[1]Station 4+95/Station 535+00			**Station 515+00**		
Top of Levee Elevation = 4866.7 feet			**Top of Levee Elevation = 4865 feet**		
WSE (feet)	**P_u**	**WSE Height above/below Levee (feet)**	**WSE (feet)**	**P_u**	**WSE Height above/below Levee (feet)**
4868	1	1.3	4866	1	1
4866.9	0.31	0.2	4865	0.11	0
4864.6	0.16	-2.1	4863	0.04	-2
4862	0	-4.7	4862	0.02	-3
			4859.5	0	-5.5
			4855	0	-10
Winslow Levee					
[2]Station 370+00			**[3]Station 290+00**		
Top of Levee Elevation = 4855.4 feet			**Top of Levee Elevation = 4852.6 feet**		
WSE (feet)	**P_u**	**WSE Height above/below Levee (feet)**	**WSE (feet)**	**P_u**	**WSE Height above/below Levee (feet)**
4857	0.5	1.6	4852	1	-0.6
4856	0.25	0.6	4849	0.83	-3.6
4853	0.14	-2.4	4847	0.52	-5.6
4852.5	0.08	-2.9	4845	0.24	-7.6
4850.5	0.02	-4.9	4840	0	-12.6
4849	0.001	-6.4			

WSE = Water Surface Elevation

P_u = Probability of Unsatisfactory Performance

WSE Height above/below Levee = WSE - Top of Levee Elevation

RWDL = Ruby Wash Diversion Levee

WL = Winslow Levee

1. RWDL Station 4+95 used as representative cross-section for Index Reach 1 in Economic Analysis. It is equivalent to WL Station 535+00.
2. WL Station 370+00 is approximately 2500 feet downstream from the Homolovi I Pueblo
3. WL Station 290+00 used as representative cross-section for Index Reach 2 in Economic Analysis
4. All stationing provided is hydraulic model stationing

Table 12: Probability of Unsatisfactory Performance vs Frequency and Discharge

Winslow Levee							
Station 535+00				**Station 515+00**			
Top of Levee Elevation = 4866.70 feet				**Top of Levee Elevation = 4865 feet**			
Frequency (% ACE)	**Discharge (cfs)**	**WSE (feet)**	**P_u**	**Frequency (% ACE)**	**Discharge (cfs)**	**WSE (feet)**	**P_u**
50	8,070	4857	0	50	8,070	4854.4	0
20	16,360	4860	0	20	16,360	4856.7	0
10	24,400	4862	0	10	24,400	4858.3	0
4	38,310	4864.6	0.16	4	38,310	4860.4	0.01
2	52,020	4866.9	0.31	2	52,020	4862.3	0.03
1	69,200	4871.7	1	1	69,200	4864.1	0.08
0.5	90,660	4872.7	1	0.5	90,660	4866.7	1
0.2	127,250	4873.2	1	0.2	127,250	4871.3	1
Winslow Levee							
Station 370+00				**Station 290+00**			
Top of Levee Elevation = 4855.38 feet				**Top of Levee Elevation = 4865 feet**			
Frequency (% ACE)	**Discharge (cfs)**	**WSE (feet)**	**P_u**	**Frequency (% ACE)**	**Discharge (cfs)**	**WSE (feet)**	**P_u**
50	8,070	4846	0	50	8,070	4840.6	0.03
20	16,360	4847.3	0	20	16,360	4842.2	0.11
10	24,400	4848.3	0	10	24,400	4843.3	0.16
4	38,310	4849.7	0.01	4	38,310	4844.8	0.23
2	52,020	4850.9	0.03	2	52,020	4846	0.38
1	69,200	4852.1	0.07	1	69,200	4847.3	0.57
0.5	90,660	4853.5	0.16	0.5	90,660	4848.7	0.78
0.2	127,250	4854.5	0.2	0.2	127,250	4850.6	0.92
WSE = Water Surface Elevation							
P_u = Probability of Unsatisfactory Performance							
WL = Winslow Levee							
RWDL = Ruby Wash Diversion Levee							
WL Station 535+00 = RWDL Station 4+95 in Geotechnical Levee Fragility Report							

Table 13: Comparison of Levee Heights for With-Project Alternatives to Baseline Condition

Alternative	WSE Profile	Reach ID	Avg. Change in Levee Height (ft)	Max. Change in Levee Height (ft)
1.1	1% ACE	Reach 1	2	2.7
1.1	1% ACE	Reach 2	1.1	1.9
1.1	1% ACE	Reach 3	2.2	3.1
1.1	1% ACE	Reach 4 and 5	0.5	1.5
3.1	1% ACE	Reach 1	1.9	2.6
3.1	1% ACE	Reach 2	1	1.8
3.1	1% ACE	Reach 3	1.9	2.9
3.1	1% ACE	Reach 4	0.25	1.5
3.1	1% ACE	Reach 5	0.5	1.6
8	1% ACE	Reach 1	2	2.7
8	1% ACE	Reach 2	1.1	1.9
8	1% ACE	Reach 3	2.2	3.1
8	1% ACE	Reach 4	0.5	1.5
8	1% ACE	Reach 5	0.5	1.6
10 & 10.1	1% ACE	Reach 1	2	2.7
10 & 10.1	1% ACE	Reach 2	0.7	1.9
10 & 10.1	1% ACE	Reach 3	2	3.1
10 & 10.1	1% ACE	Reach 4	0.5	1.6
10 & 10.1	1% ACE	Reach 5	0.9	1.5
10.2	4% ACE	Reach 1	2.1	3
10.2	4% ACE	Reach 2	0	0.1
10.2	4% ACE	Reach 3	0	0.4
10.2	4% ACE	Reach 4	0	0
10.2	4% ACE	Reach 5	0	0
10.3	2% ACE	Reach 1	0.6	1.3
10.3	2% ACE	Reach 2	0	1.1
10.3	2% ACE	Reach 3	0.5	1.7
10.3	2% ACE	Reach 4	0	0.2
10.3	2% ACE	Reach 5	0	0.1
10.4	0.5% ACE	Reach 1	2.9	3.7
10.4	0.5% ACE	Reach 2	0.5	1.6
10.4	0.5% ACE	Reach 3	2.7	3.8
10.4	0.5% ACE	Reach 4	1.5	3.1
10.4	0.5% ACE	Reach 5	2.4	3

Notes:

Reach 1: Upstream from BNSF Railroad Bridge (hydraulic river station 542+50 to 529+83)

Reach 2: BNSF Railroad Bridge to I-40 Bridges (hydraulic river station 528+87 to 505+03)

Reach 3: New Levee Section along I-40 (compared to I-40 embankment, which is not classified as a levee) - hydraulic river station 495+00 to 475+00

Reach 4: From I-40 to Setback Levee (hydraulic river station 475+00 to 365+00)

Reach 5: Setback Levee to End of Levee - hydraulic river station 365+00 to 320+00 (Alternatives 10 and higher) or 190+00 (Alternatives 1.1, 3.1 and 8)

Table 14: Slope Protection Recommendations for With-Project Alternatives

Alt. #	Reach	Reach Description	Upstream Station	Downstream Station	Length (ft)	Slope Protection
1.1	1	Upstream from Route 66 Bridge	N/A	524+93	3300	24-inch riprap
1.1	2	Route 66 Bridge to I-40 Bridges	524+93	505+03	2000	42-inch riprap[1]
1.1	3	New Levee Along I-40	505+03	475+00	3750	24-inch riprap
1.1	4	From I-40 to South River Impingement	475+00	420+00	5000	24-inch riprap
1.1	5	South River Impingement	420+00	360+00	6300	Soil Cement[1]
1.1	6	Between the 2 Impingement Locations	360+00	285+00	6100	24-inch riprap
1.1	7	Northern River Impingement	285+00	235+00	3350	Soil Cement[1]
1.1	8	North Impingement to End of Levee	235+00	190+00	6800	24-inch riprap
3.1	1	Upstream from Route 66 Bridge	N/A	524+93	3300	24-inch riprap
3.1	2	Route 66 Bridge to I-40 Bridges	524+93	505+03	2000	42-inch riprap[1]
3.1	3	New Levee Along I-40	505+03	475+00	3750	24-inch riprap
3.1	4	New Setback Levee	475+00	325+00	13050	24-inch riprap
3.1	N/A	*South River Impingement*[2]	*420+00*	*360+00*	*5800*	*Soil Cement*[3]
3.1	5	Setback Levee to North Impingement	325+00	285+00	2950	24-inch riprap
3.1	6	Northern River Impingement	285+00	235+00	3350	Soil Cement[1]
3.1	7	North Impingement to End of Levee	235+00	190+00	6800	24-inch riprap
8	1	Upstream from Route 66 Bridge	N/A	524+93	3300	24-inch riprap
8	2	Route 66 Bridge to I-40 Bridges	524+93	505+03	2000	42-inch riprap[1]
8	3	New Levee Along I-40	505+03	475+00	3750	24-inch riprap
8	4	From I-40 to South River Impingement	475+00	420+00	5000	24-inch riprap
8	5	South Impingement (Setback Levee)	420+00	360+00	5800	Soil Cement[1]
8	6	Between the 2 Impingement Locations	360+00	285+00	6100	24-inch riprap
8	7	Northern River Impingement	285+00	235+00	3350	Soil Cement[1]
8	8	North Impingement to End of Levee	235+00	190+00	6800	24-inch riprap
10/10.1	1	Upstream from Route 66 Bridge	N/A	524+93	3300	24-inch riprap
10/10.1	2	Route 66 Bridge to End of I-40 Levee	524+93	505+03	2000	42-inch riprap[1]
10/10.1	3	New Levee Along I-40	505+03	475+00	3750	24-inch riprap
10/10.1	4	From I-40 to South River Impingement	475+00	420+00	5000	24-inch riprap
10/10.1	5	South River Impingement	420+00	360+00	5800	Soil Cement[1]
10/10.1	6	South Impingement to End of Levee	360+00	320+00	3000	24-inch riprap

Notes:

1. Indicates that Soil Cement or Grouted Stone could be used
2. Indicates that Soil Cement or Grouted Stone may be used on the impingement area (even with the setback levee)

- Hydraulic River Station provided
- 24-inch riprap indicates a D_{100} of 24 inches and a layer thickness of 24 inches (ungrouted)
- Slope protection recommendations based on hydraulic analyses results

Table 15: Comparison of Floodplain Results for FLO-2D Hydraulic Analyses (Homolovi I Pueblo)

			Compared to Alternative 10.1	
Hydraulic Model Description	**Maximum Flow Depth[5]**	**Average Velocity**	**Change in WSE[6]**	**Average Change in Floodplain Extent[7]**
	(feet)	**(feet/s)**	**(feet)**	**(feet)**
[1]Baseline (No Failure) - Scenario 1	5.1	3.4	0.2	2
[2]Baseline (Levee Failure) - Scenario 3	4.4	3.1	-0.5	-10
[3]Baseline (Levee Failure) - Scenario 4	4.5	3	-0.4	-8
[4]Baseline (Levee Failure) - Scenario 5	4.9	3.2	0.0	0
Alternative 10/10.1	4.9	3.2	N/A	N/A
Alternative 3.1	2.9	3.1	-2.0	-25

Notes

1. The baseline condition no levee failure scenario assumes that the WSE reaches the maximum height for the 1 % ACE flood event

2. The baseline condition levee failure (Scenarios 3) assumes that the levee fails at four locations along the levee (1 U/S & 3 D/S)

3. The baseline condition levee failure (Scenarios 4) assumes that the levee fails upstream near State Route 87 Bridge

4. The baseline condition levee failure (Scenarios 5) assumes that the levee fails at three locations downstream of Homolovi I Pueblo

5. Maximum flow depth for the 1% ACE at Homolovi I Pueblo

6. The Change in WSE compared to Alternative 10.1 (Positive Number equals increase in height)

7. The average change in floodplain extent is the horizontal difference in inundated area at Homolovi I Pueblo

Table 16: Comparison of With-Project Alternatives to Baseline Condition

Alternative	WSE Profile	Reach Description	Upstream Station	Downstream Station	Avg. Change in WSE[1]	Avg. Main Channel Velocity[2]
					(feet)	(feet/second)
Baseline	1% ACE	Upstream from BNSF Bridge	560+00	529+83	N/A	3.4
Baseline	1% ACE	BNSF Bridge to I-40 WB Bridge	529+83	504+71	N/A	8
Baseline	1% ACE	Downstream from I-40 WB Bridge	504+71	400+00	N/A	7.5
Baseline	1% ACE	Near Homolovi I Pueblo	400+00	390+00	N/A	7.5
Baseline	1% ACE	Near Homolovi to Station 320+00	385+00	320+00	N/A	5.6
1.1	1% ACE	Upstream from BNSF Bridge	560+00	529+83	-2.3	6.3
1.1	1% ACE	BNSF Bridge to I-40 WB Bridge	529+83	504+71	-0.4	6.7
1.1	1% ACE	Downstream from I-40 WB Bridge	504+71	400+00	0	7.5
1.1	1% ACE	Near Homolovi I Pueblo	400+00	390+00	-0.1	7.8
1.1	1% ACE	Near Homolovi to Station 320+00	385+00	320+00	0	5.5
3.1	1% ACE	Upstream from BNSF Bridge	560+00	529+83	-2.4	6.4
3.1	1% ACE	BNSF Bridge to I-40 WB Bridge	529+83	504+71	-0.5	6.8
3.1	1% ACE	Downstream from I-40 WB Bridge	504+71	400+00	-0.7	7.5
3.1	1% ACE	Near Homolovi I Pueblo	400+00	390+00	-0.7	7.3
3.1	1% ACE	Near Homolovi to Station 320+00	385+00	320+00	-0.3	5
8/ 10 / 10.1	1% ACE	Upstream from BNSF Bridge	560+00	529+83	-2.3	6.3
8/ 10 / 10.1	1% ACE	BNSF Bridge to I-40 WB Bridge	529+83	504+71	-0.4	6.7
8/ 10 / 10.1	1% ACE	Downstream from I-40 WB Bridge	504+71	400+00	0	7.5
8/ 10 / 10.1	1% ACE	Near Homolovi I Pueblo	400+00	390+00	-0.2	7.7
8/ 10 / 10.1	1% ACE	Near Homolovi to Station 320+00	385+00	320+00	0	5.4
10.2	4% ACE	Upstream from BNSF Bridge	560+00	529+83	0	4.4
10.2	4% ACE	BNSF Bridge to I-40 WB Bridge	529+83	504+71	0	7.6
10.2	4% ACE	Downstream from I-40 WB Bridge	504+71	400+00	0	6.5
10.2	4% ACE	Near Homolovi I Pueblo	400+00	390+00	-0.1	6.1
10.2	4% ACE	Near Homolovi to Station 320+00	385+00	320+00	0	4.4
10.3	2% ACE	Upstream from BNSF Bridge	560+00	529+83	-3.2	6.3
10.3	2% ACE	BNSF Bridge to I-40 WB Bridge	529+83	504+71	-1	6.3
10.3	2% ACE	Downstream from I-40 WB Bridge	504+71	400+00	0	6.9
10.3	2% ACE	Near Homolovi I Pueblo	400+00	390+00	-0.1	7
10.3	2% ACE	Near Homolovi to Station 320+00	385+00	320+00	0	4.9
10.4	0.5% ACE	Near Homolovi I Pueblo[4]	400+00	390+00	-0.2	N/A

Notes:

1. Compared to Baseline Condition ACE Flood (Negative Value incicates drop in WSE)
2. Main Channel Velocity is often higher than the velocity in the overbank areas
3. HEC-RAS was used to determine average changes in WSE and average main channel velocities
4. FLO-2D was used to determine difference in WSE and velocity due to volume conservation
5. Average change in WSE are compared to the baseline condition of comparable frequecy (i.e. 1% to 1%, 2% to 2%, 4% to 4%)

Table 17: Project Performance and Conditional Non-Exceedance Probabilities

Alternative	Reach Identifier	Target Annual Exceedance Probability		Long Term Risk (Year Period)			Conditional Non-Exceedance Probability (Organized by Design Event - Annual Chance Exceedance)			
		Median	Expected	10	30	50	10% ACE	2% ACE	1% ACE	0.2% ACE
Without Project	1	0.0380	0.0410	0.3420	0.7150	0.8766	0.9337	0.2629	0.0715	0.0025
	2	0.0692	0.0696	0.5137	0.8850	0.9728	0.8781	0.6728	0.5058	0.1837
1.1	1	0.0037	0.0049	0.0482	0.1377	0.2188	1.000	0.9877	0.8804	0.2702
	2	0.0022	0.0012	0.0122	0.0362	0.0596	1.000	0.9944	0.9478	0.6059
3.1	1	0.0039	0.0051	0.0498	0.1421	0.2254	1.000	0.9863	0.8730	0.2593
	2	0.0022	0.0012	0.0122	0.0362	0.0596	1.000	0.9944	0.9478	0.6059
7	1	0.0380	0.0410	0.3420	0.7150	0.8766	0.9337	0.2629	0.0715	0.0025
	2	0.0692	0.0696	0.5143	0.8854	0.9730	0.8780	0.6717	0.5043	0.1821
8	1	0.0037	0.0049	0.0482	0.1377	0.2188	1.000	0.9877	0.8804	0.2702
	2	0.0022	0.0012	0.0122	0.0362	0.0596	1.000	0.9944	0.9478	0.6059
9	1	0.0290	0.0323	0.2799	0.6265	0.8063	0.9952	0.2954	0.0808	0.0029
	2	0.0692	0.0697	0.5143	0.8854	0.9730	0.8780	0.6717	0.5043	0.1821
10	1	0.0037	0.0049	0.0482	0.1161	0.2187	1.000	0.9879	0.8808	0.2696
	2	0.0692	0.0695	0.5137	0.8850	0.9728	0.8780	0.6716	0.5051	0.2120
10.1	1	0.0037	0.0049	0.0482	0.1161	0.2187	1.000	0.9879	0.8808*	0.2696
	2	0.0692	0.0695	0.5137	0.8850	0.9728	0.8780	0.6716	0.5051	0.2120
10.2	1	0.0137	0.0163	0.1516	0.3894	0.5605	1.000	0.7092	0.3523	0.0261
	2	0.0692	0.0695	0.5131	0.8846	0.9726	0.8782	0.6732	0.5060	0.2125
10.3	1	0.0065	0.0085	0.0815	0.2251	0.3462	1.000	0.9323	0.6944	0.1201
	2	0.0692	0.0695	0.5132	0.8847	0.9727	0.8781	0.6725	0.5064	0.2130
10.4	1	0.0024	0.0018	0.0813	0.0539	0.0882	1.000	0.9980	0.9619	0.5366
	2	0.0692	0.0695	0.5132	0.8847	0.9727	0.8781	0.6726	0.5066	0.2133

*Additional analysis has been conducted indicating that just 0.3 feet of additional height will increase the assurance to a 90% level for the 1% ACE event. Total Levee Height above water surface profile would be 3.3 feet.

Table 18: Comparison of Flood Risk Reduction Measures (FRRM) at Homolovi I Pueblo

FRRM #	WSE Profile	Upstream Station	Downstream Station	Avg. Change in WSE[1] (feet)
1	1% ACE	400+00	390+00	-0.7
2	1% ACE	400+00	390+00	-1.3
3	1% ACE	400+00	390+00	-1.3
4	1% ACE	400+00	390+00	-2.1
5	1% ACE	400+00	390+00	-2.5
6	1% ACE	400+00	390+00	-1.7
Notes				
1. Compared to Baseline Condition 1% ACE Flood (Negative Value indicates drop in WSE)				
FRRM = Flood Risk Reduction Measure				

Table 19: Interior Drainage along the Winslow Levee

Culvert Name	K-3 Channel & Floodgates	I-4 Channel & Floodgates
Status	Existing	Existing
Culvert Type/Material	Reinforced Concrete Box	Reinforced Concrete Box
Culvert Length	256 feet	54 feet, 10 in
# Box Culverts	3 RCB	4 RCB
Box Dimensions	10 feet wide x 4 feet high	10 feet wide x 4 feet high
Approx. Drainage Area	510 Acres	695 Acres
Approx. Flow Capacity	1000 cfs	1200 cfs
Location (As-Builts)	59+68+/-	92+13.41

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PLATES

LITTLE COLORADO RIVER AT WINSLOW
HYDRAULIC AND SEDIMENTATION APPENDIX
APRIL 2016

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Legend

- AZ Cities
- Little Colorado River
- Arizona Main Rivers
- Little Colorado River Watershed

N

0 20 40 80 Miles

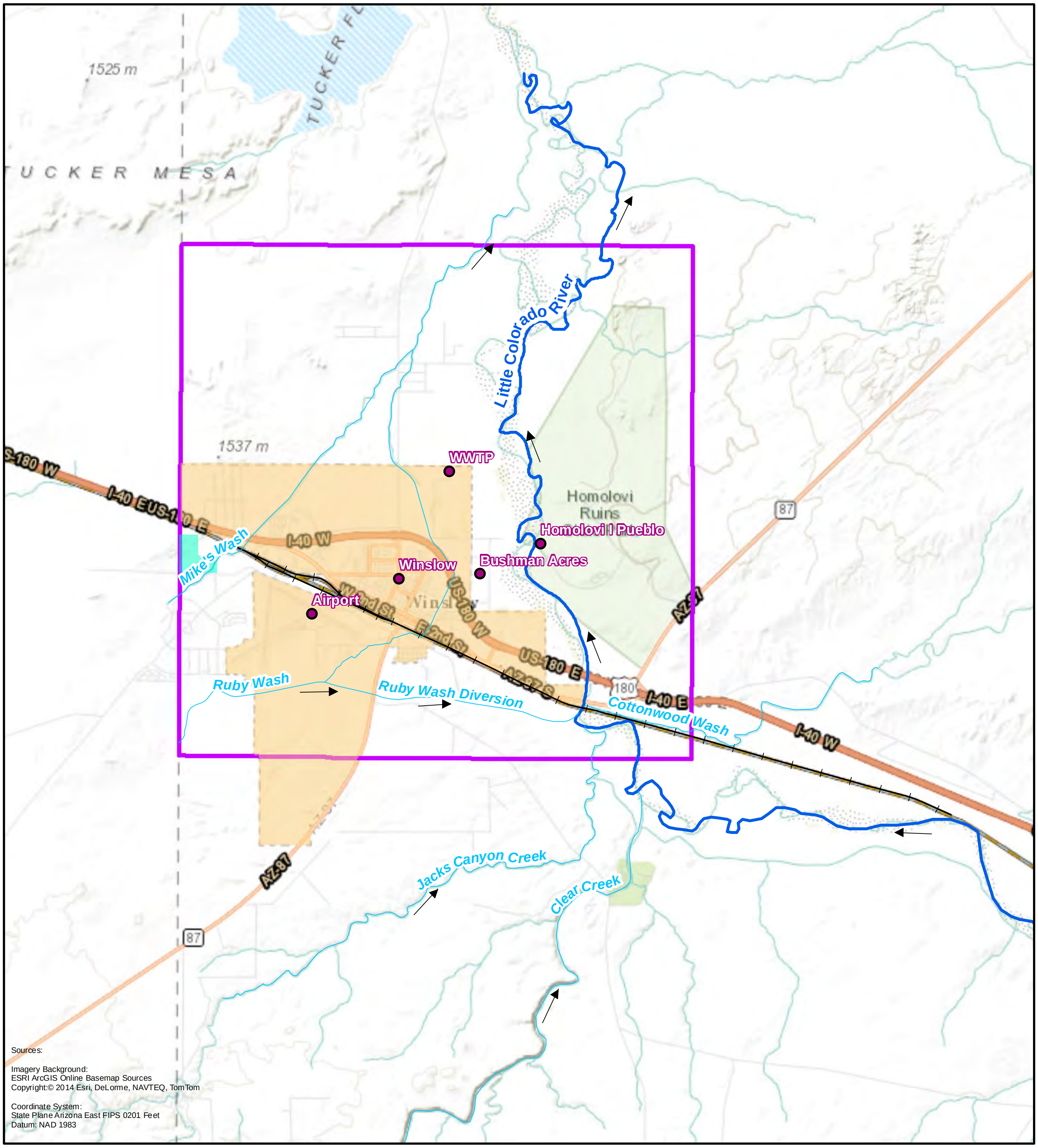
1 in = 40 miles

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

LITTLE COLORADO RIVER
WATERSHED

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 1



Legend

- Important Locations
- LCR Tributaries
- BNSF Railroad
- Little Colorado River
- Hopi Reservation
- City of Winslow
- Little Colorado River Watershed
- Study Boundary

N

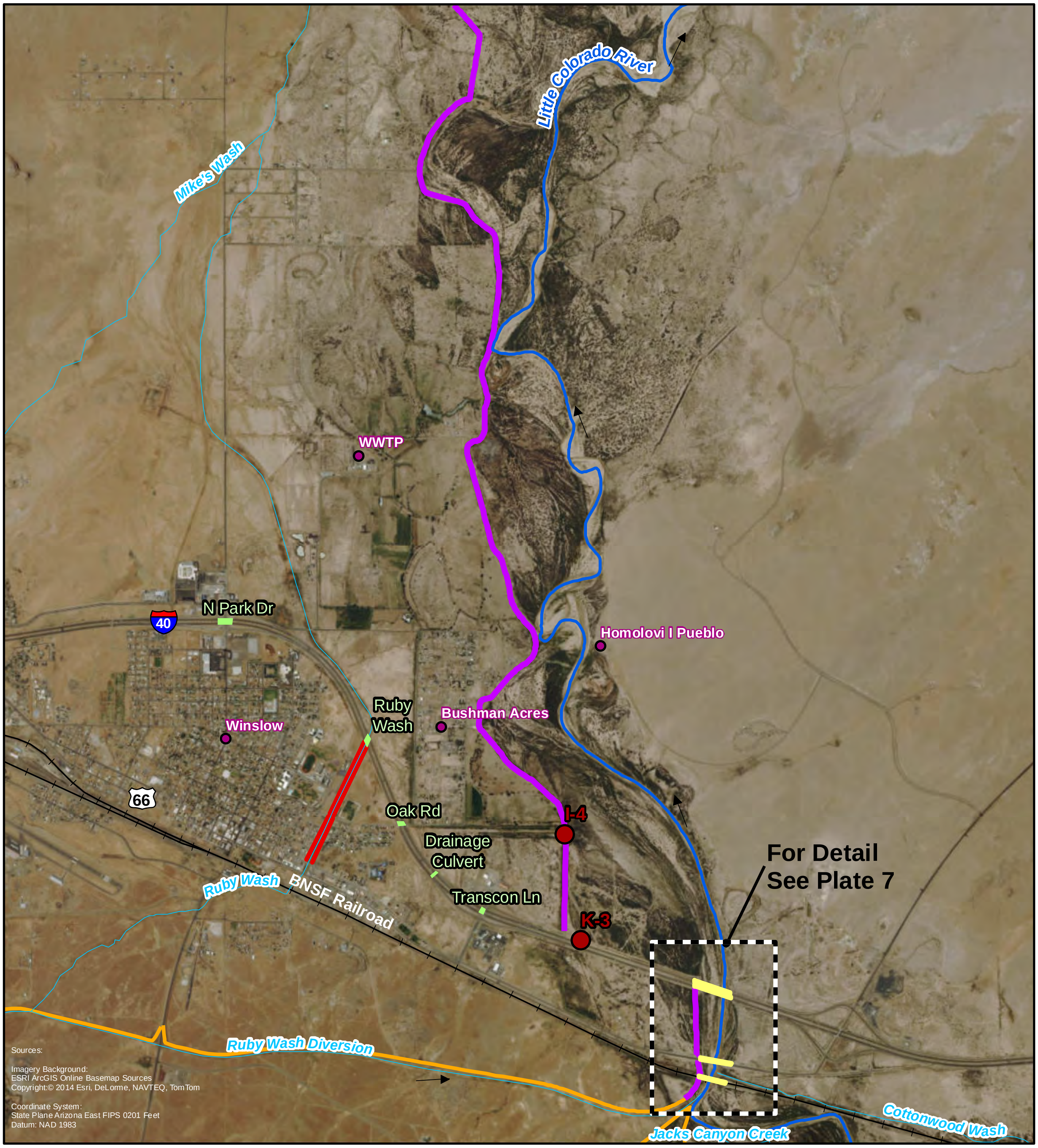
0 1 2 4 Miles

1 in = 2 miles

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

LITTLE COLORADO RIVER
STUDY AREA

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Bridges
- Interstate-40 Overpasses
- Hydraulic Gates
- Little Colorado River
- LCR Tributaries
- Winslow Levee (Existing)
- RWDL (Existing)
- Ruby Wash Levees
- BNSF Railroad

N

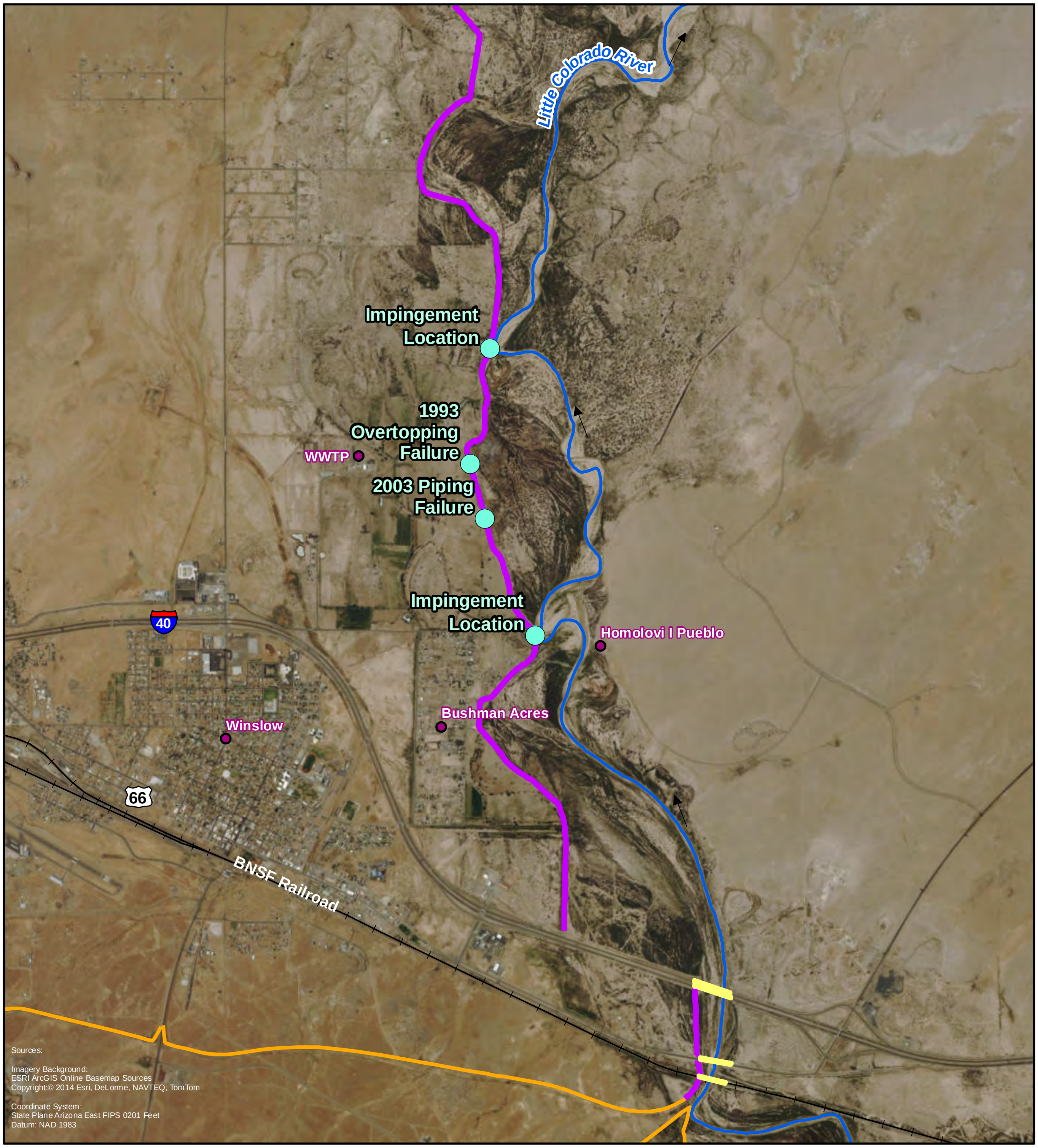
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

LITTLE COLORADO RIVER
HYDRAULIC FEATURES

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Winslow Levee Failure Locations
- Bridges
- Little Colorado River
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad

N

0 2,000 4,000 8,000 Feet

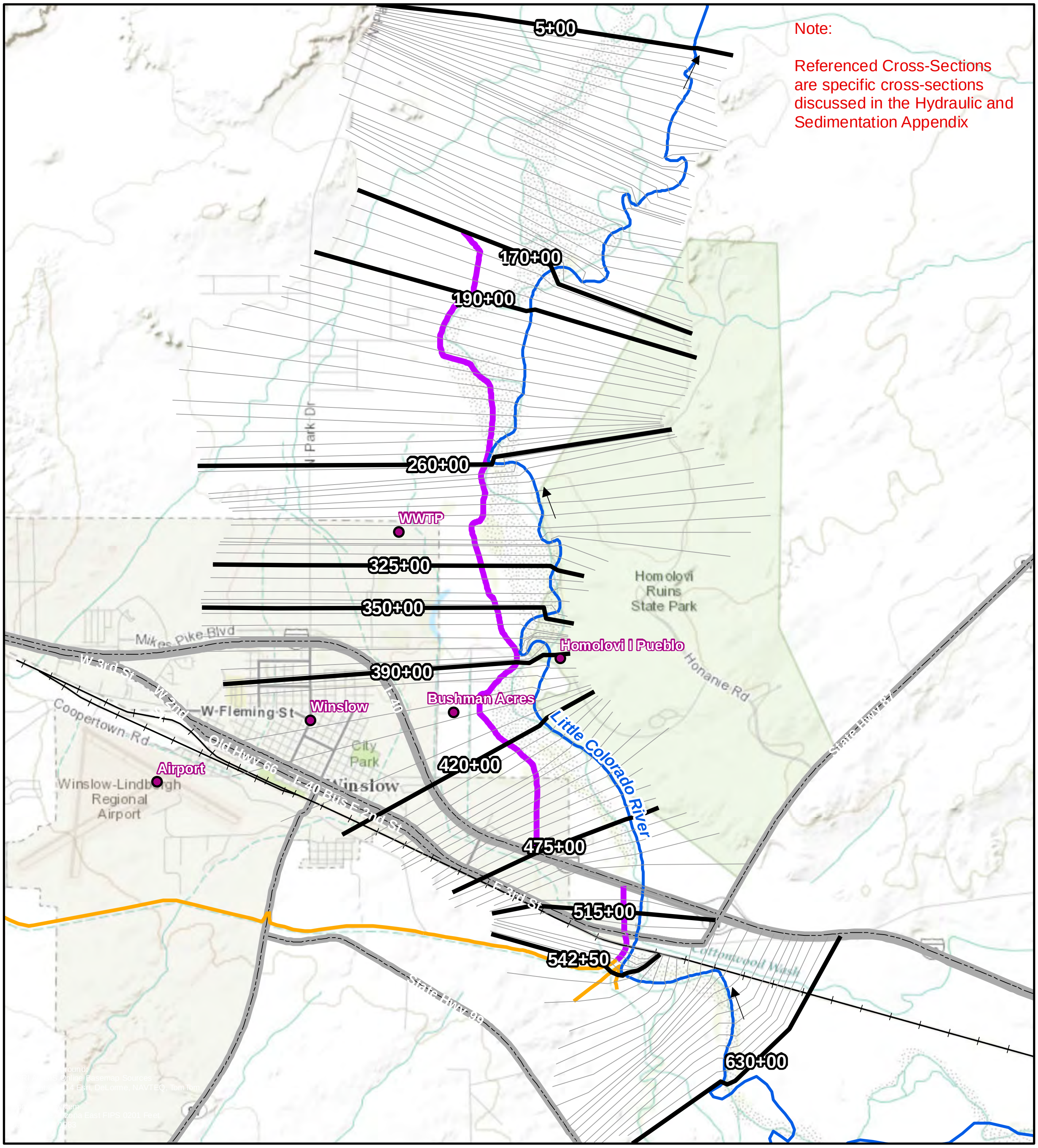
1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

WINSLOW LEVEE FLOOD HISTORY MAP

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 4



Legend

- Important Locations
- Referenced Cross-Sections
- Cross-Sections
- Little Colorado River
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad

N

0 3,000 6,000 12,000 Feet

1 in = 6,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

HEC-RAS
CROSS-SECTIONS

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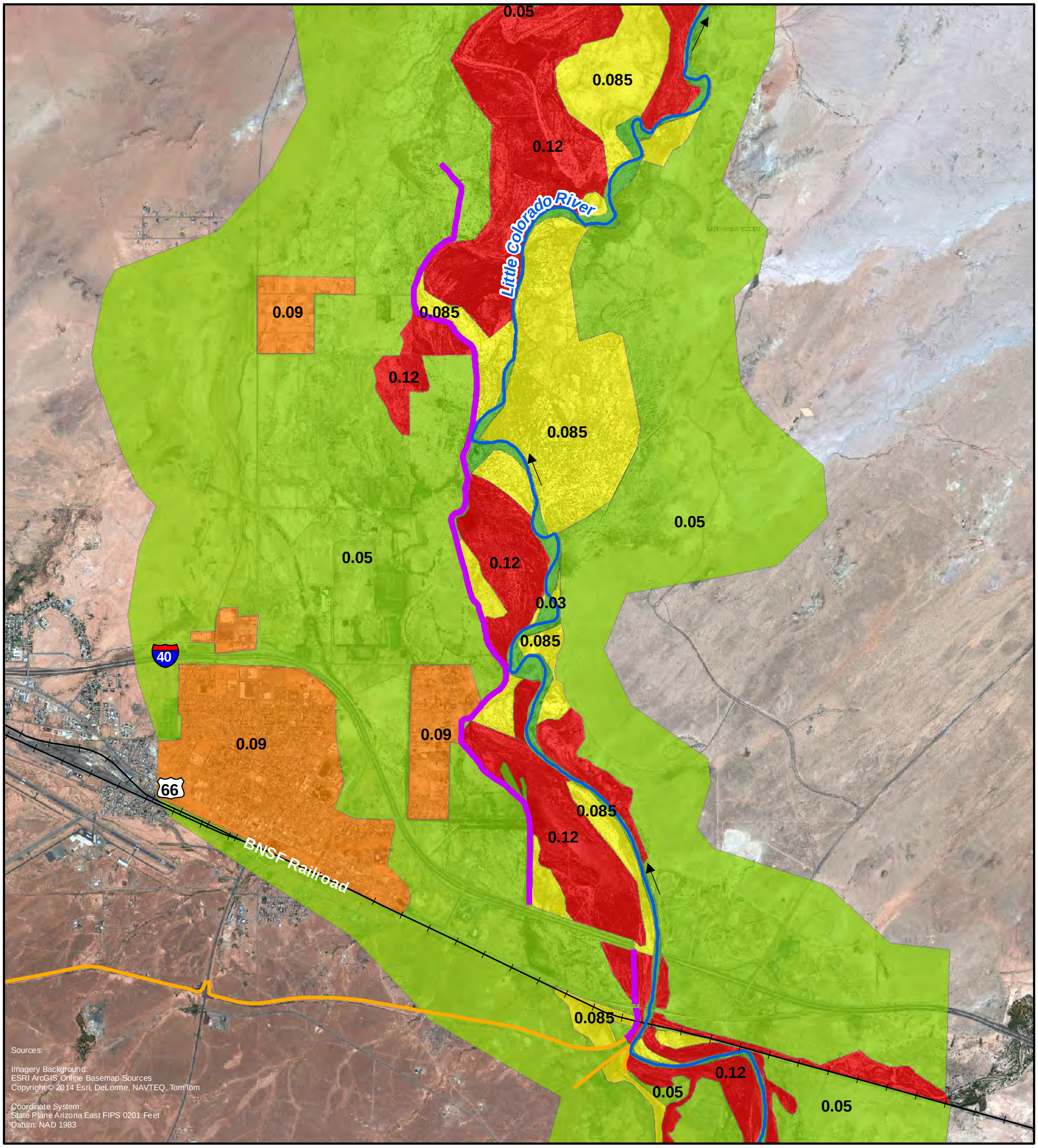
LOS ANGELES DISTRICT



Legend

N-Value

- 0.03
- 0.05
- 0.085
- 0.09
- 0.12

- Little Colorado River
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad

N

0 2,500 5,000 10,000 Feet

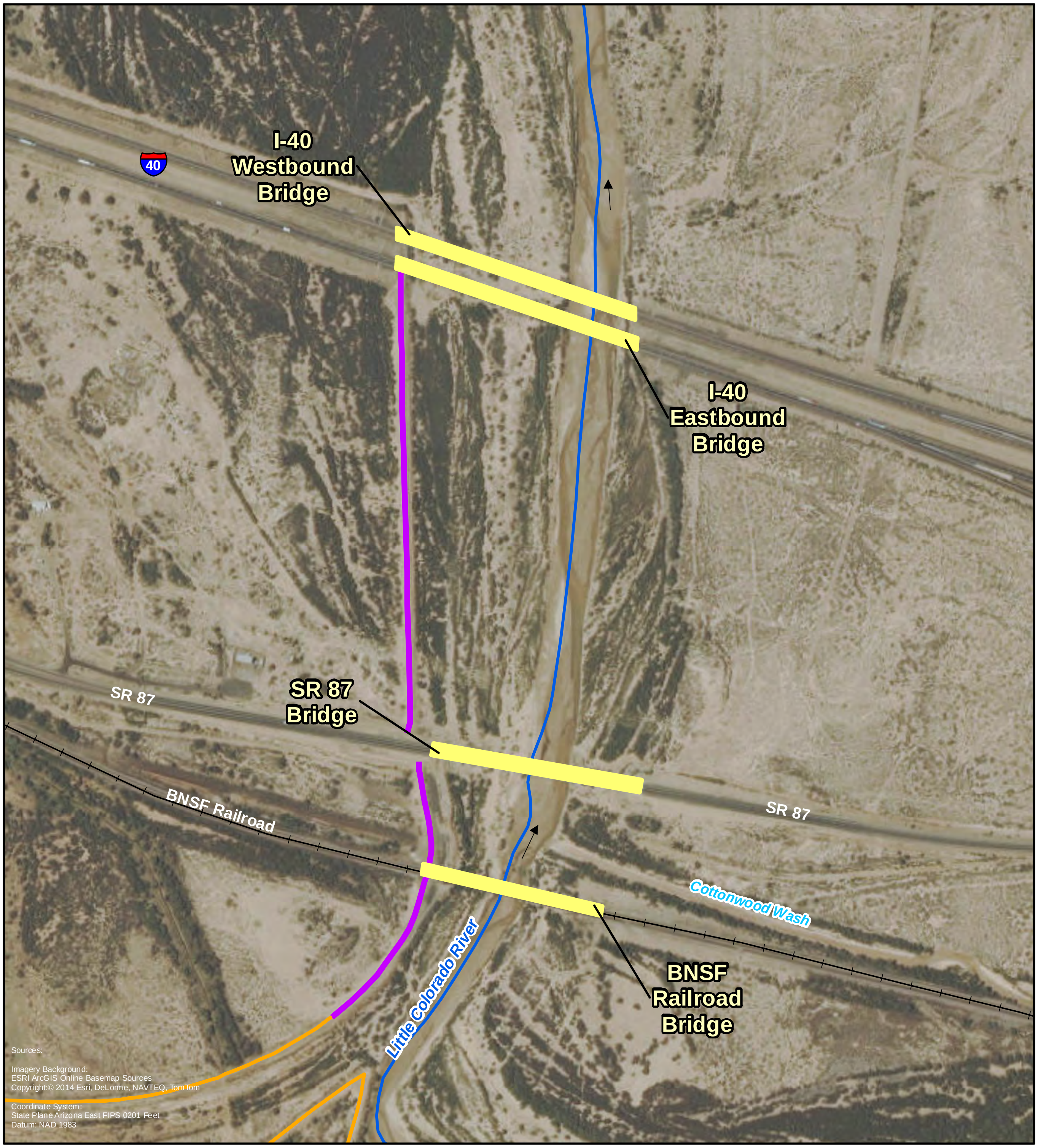
1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

MANNINGS'S
N - VALUES

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 6



Legend

- Study Area Bridges
- Little Colorado River
- RWDL (Existing)
- Winslow Levee (Existing)
- BNSF Railroad

N

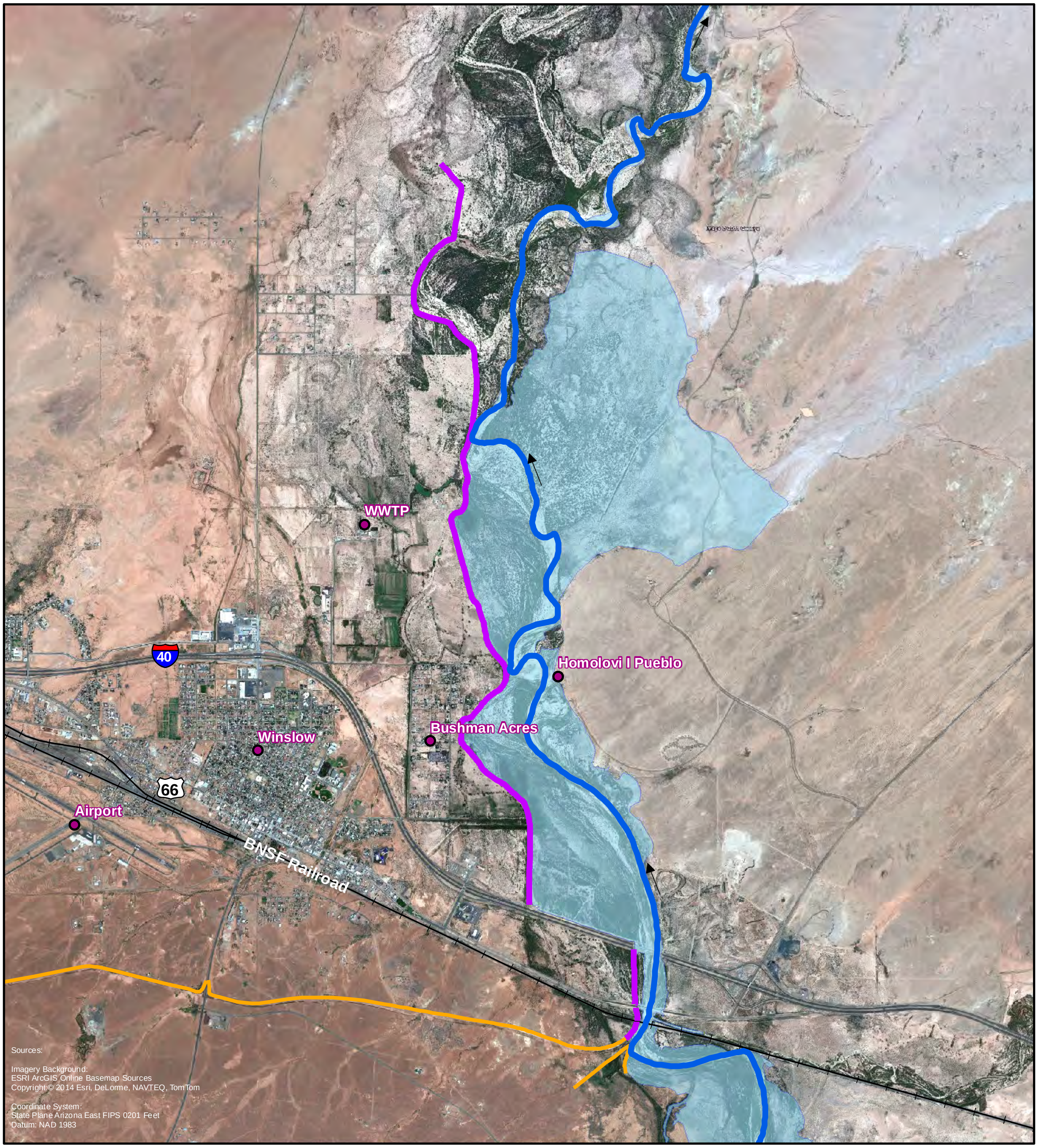
0 300 600 1,200 Feet

1 in = 600 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

STUDY AREA BRIDGE
LOCATION MAP

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad
- 50% ACE Flood

N

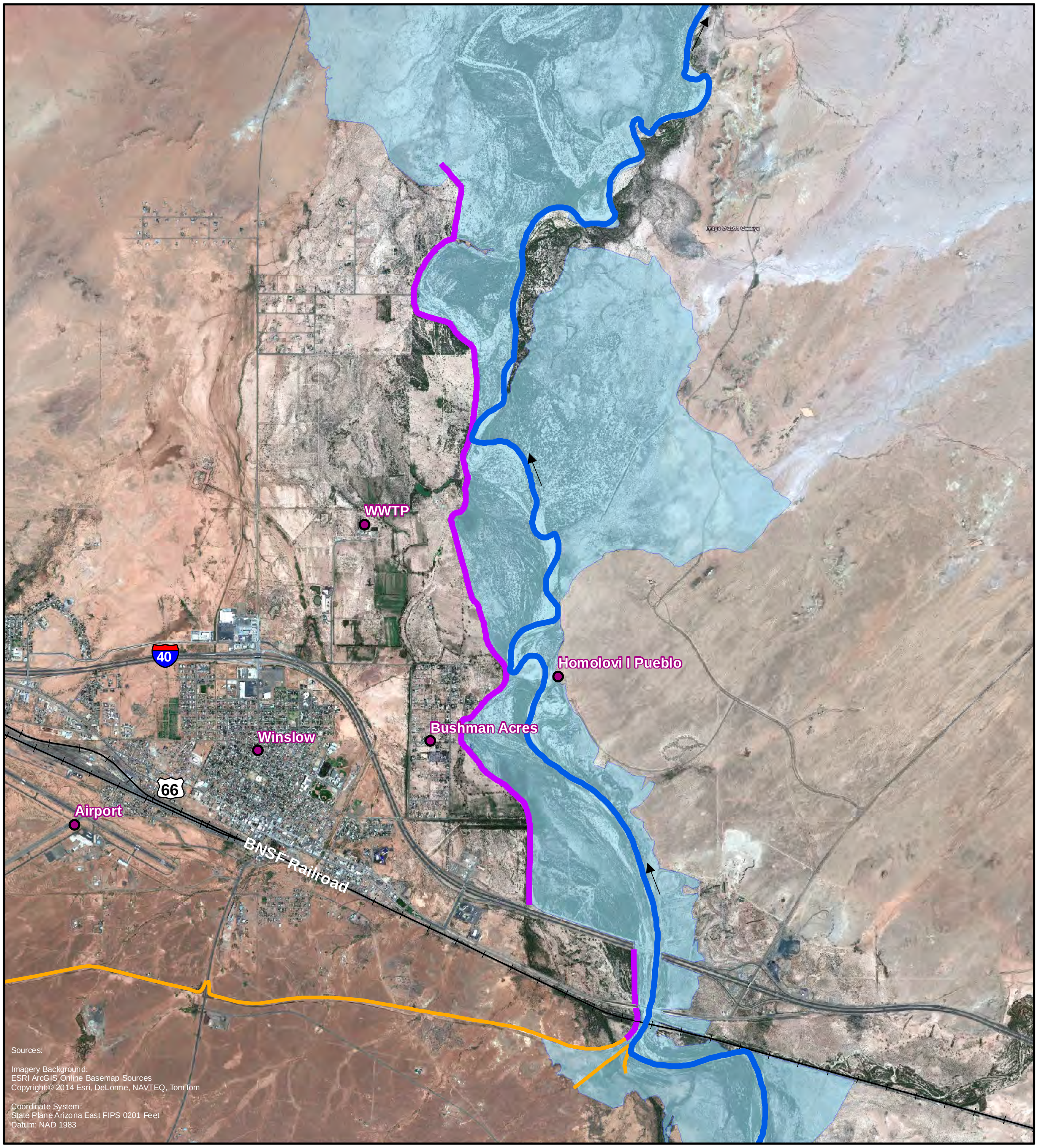
0 2,500 5,000 10,000 Feet

1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

BASELINE CONDITION
50% ACE FLOOD
(HEC-RAS)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad
- 20% ACE

N

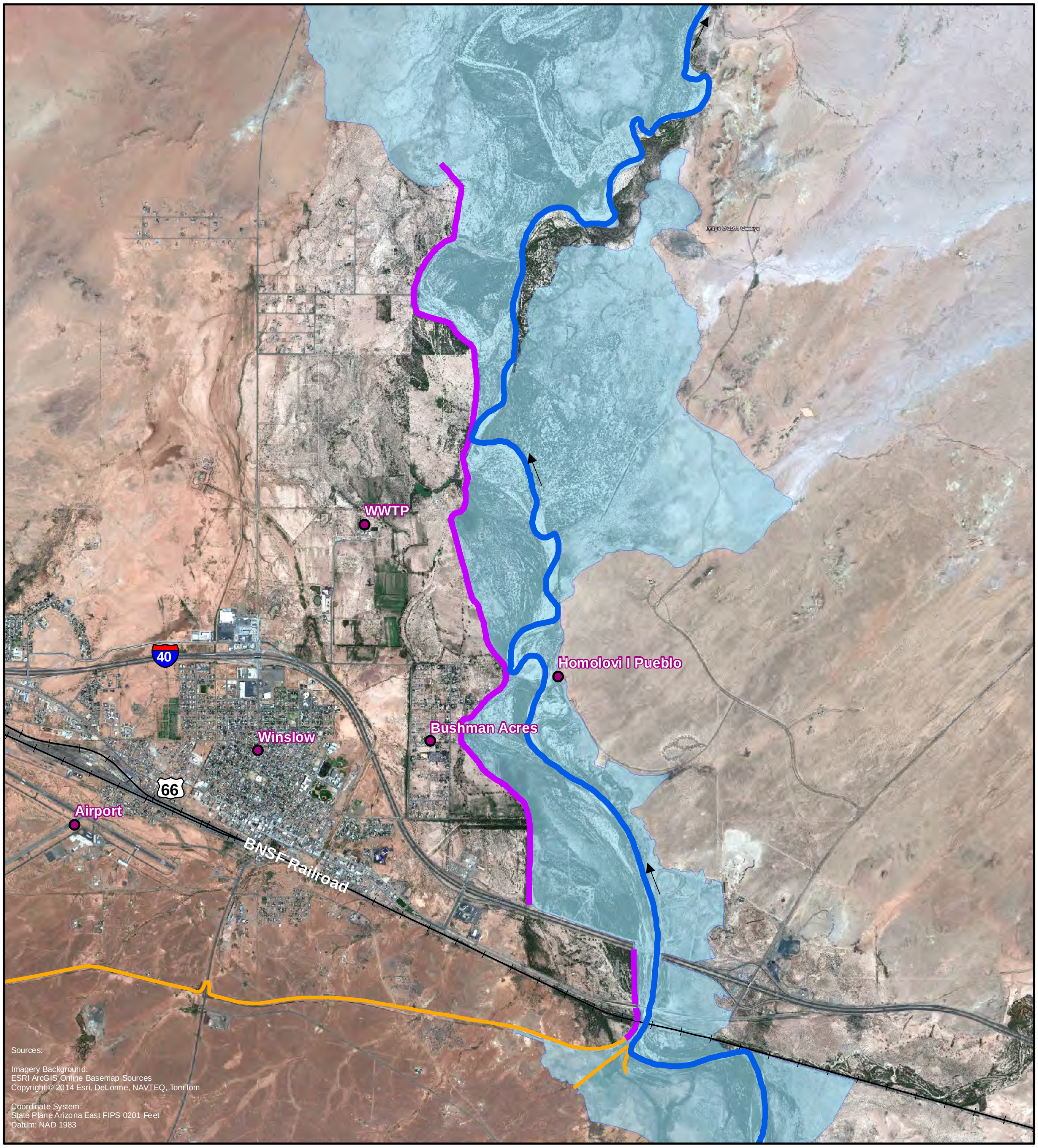
0 2,500 5,000 10,000 Feet

1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

BASELINE CONDITION
20-PERCENT ACE FLOOD
(HEC-RAS)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad
- 10% ACE Flood

N

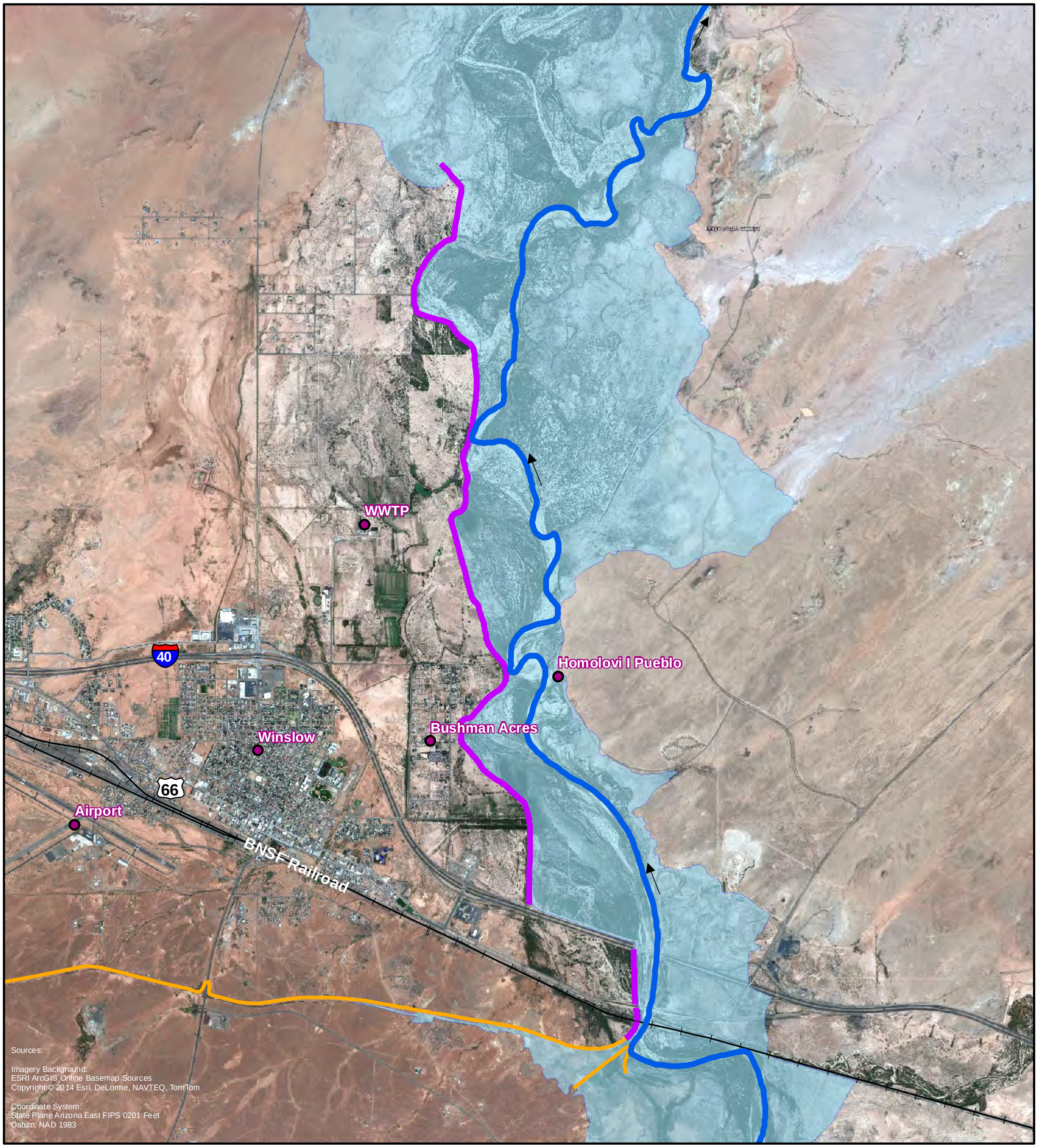
0 2,500 5,000 10,000 Feet

1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

BASELINE CONDITION
10% ACE FLOOD
(HEC-RAS)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad
- 4% ACE Flood

N

0 2,500 5,000 10,000 Feet

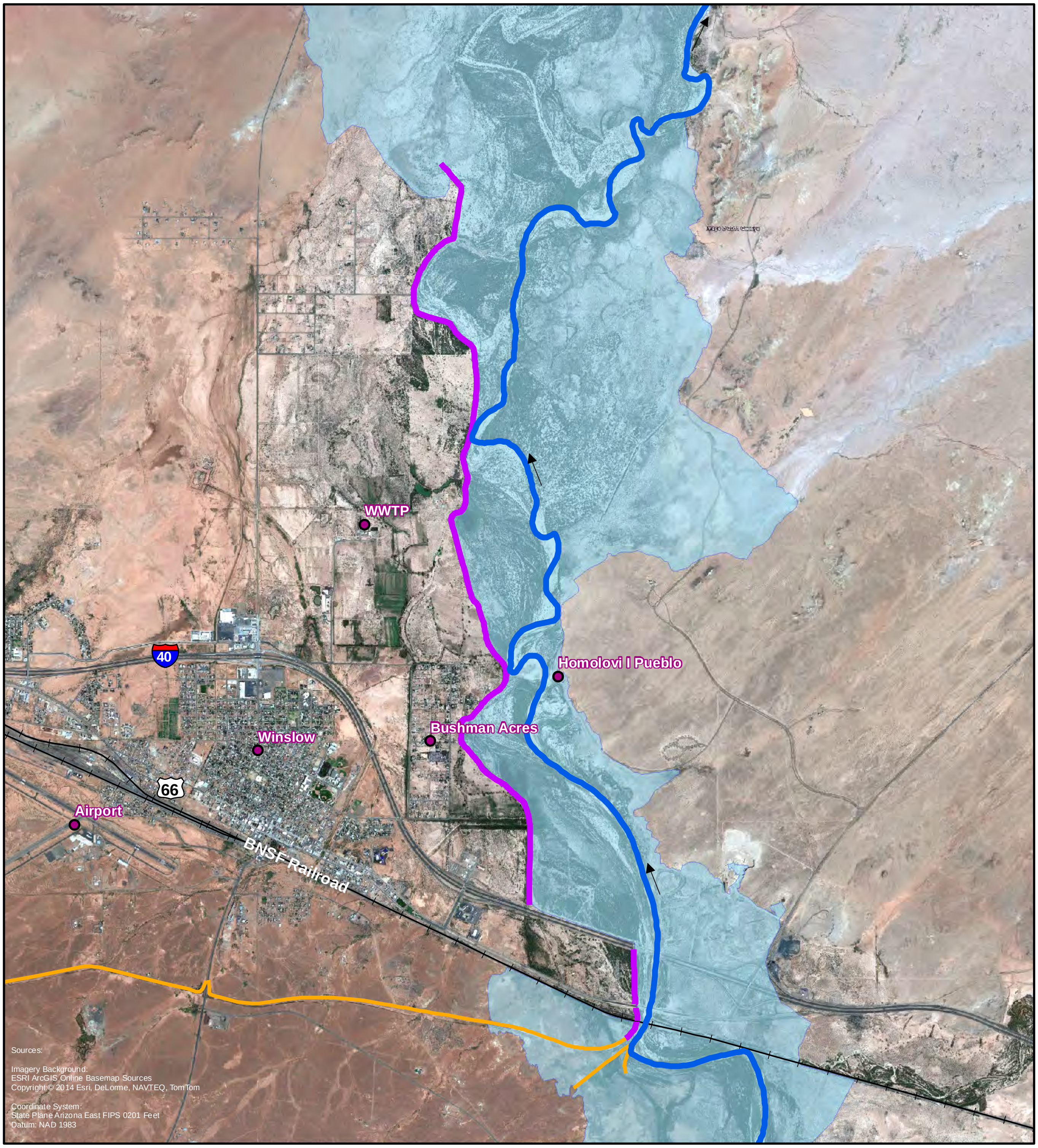
1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

BASELINE CONDITION
4% ACE FLOOD
(HEC-RAS)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 11



Legend

- Little Colorado River
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad
- 2% ACE Flood

N

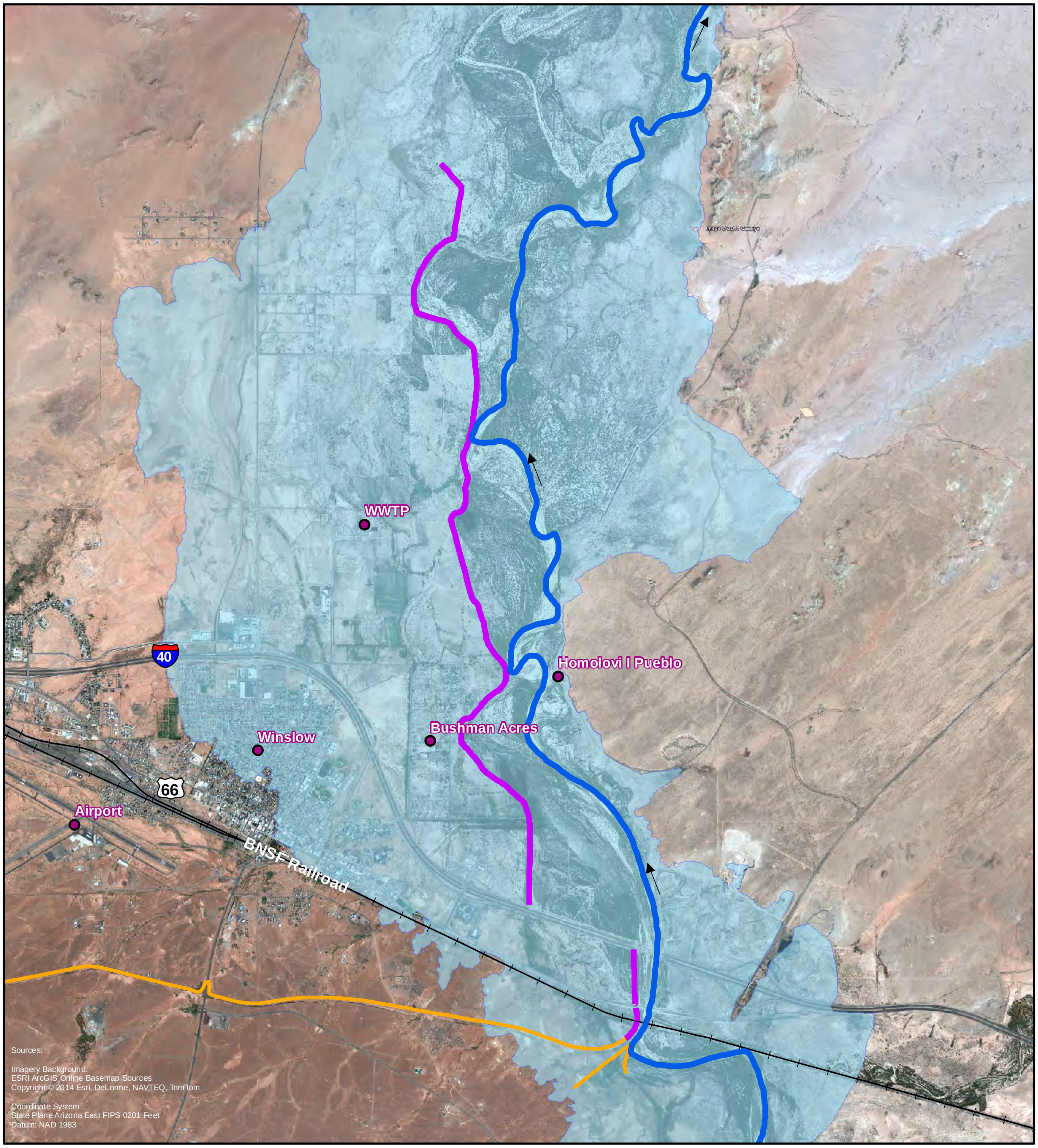
0 2,500 5,000 10,000 Feet

1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

BASELINE CONDITION
2% ACE FLOOD
(HEC-RAS)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad
- 1% ACE Flood

N

0 2,500 5,000 10,000 Feet

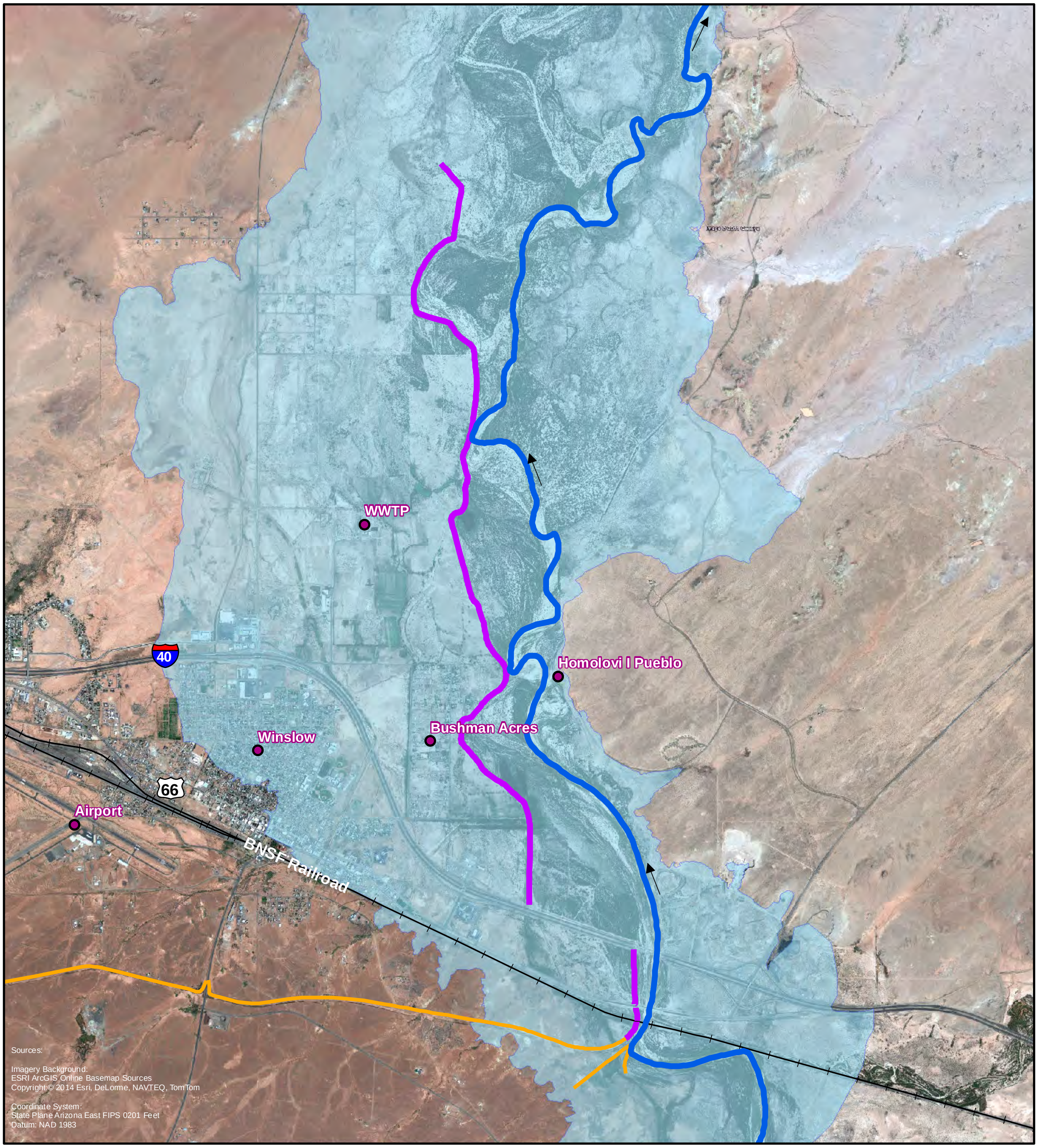
1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

BASELINE CONDITION
1% ACE FLOOD
(HEC-RAS)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 13



Legend

- 0.5% ACE Flood
- Little Colorado River
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad

N

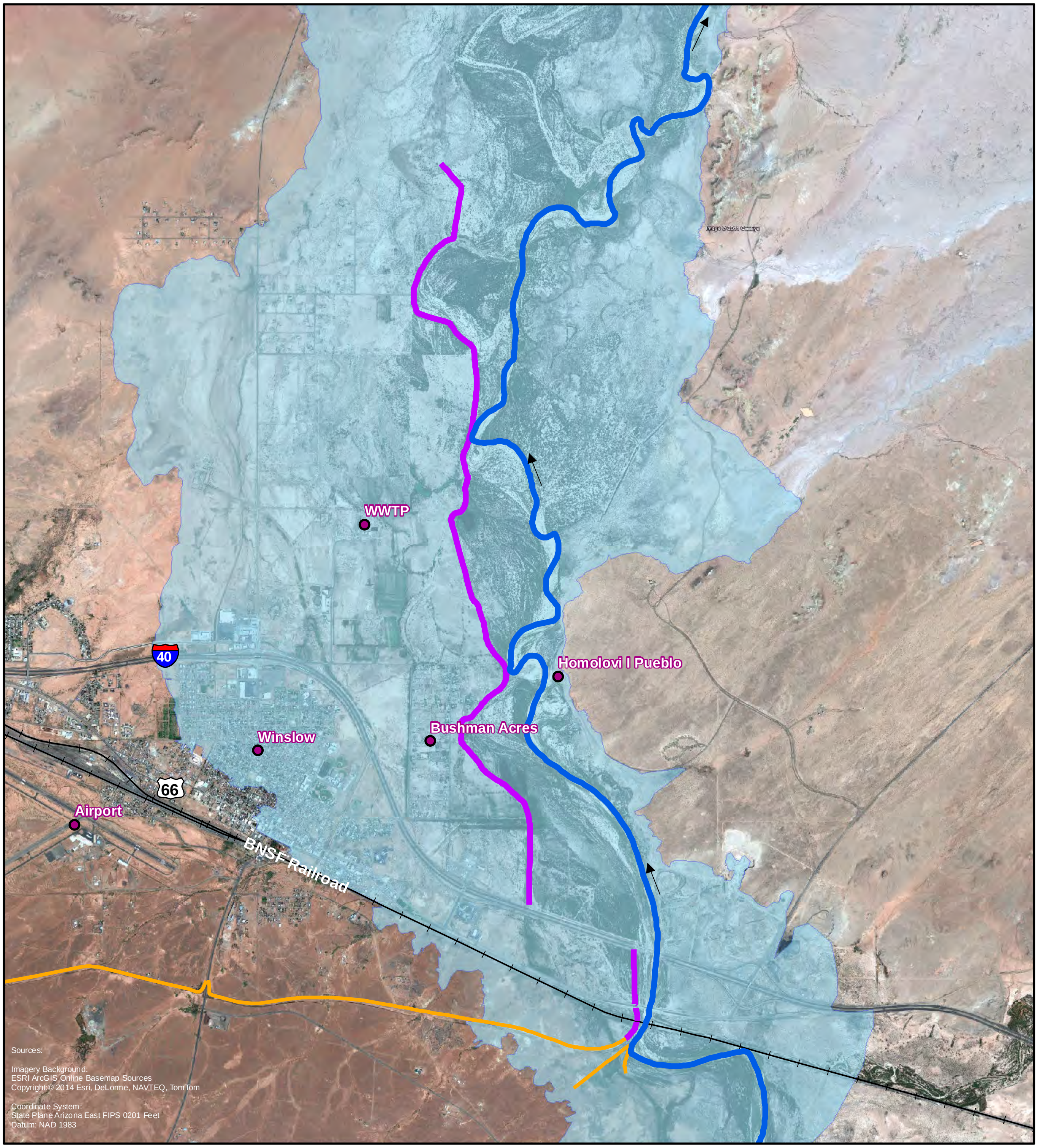
0 2,500 5,000 10,000 Feet

1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

BASELINE CONDITION
0.5% ACE FLOOD
(HEC-RAS)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- 0.2% ACE Flood
- Little Colorado River
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad

N

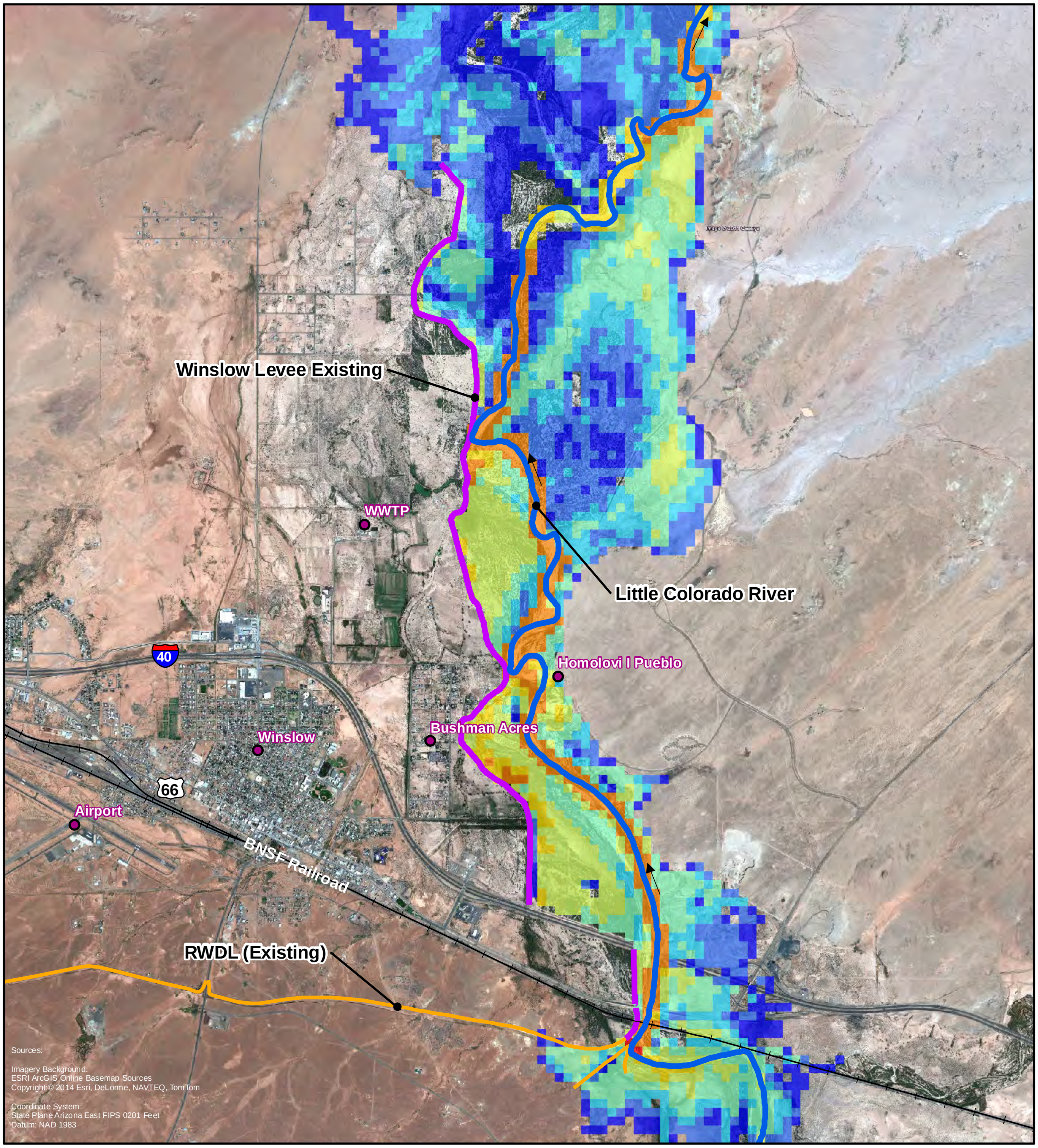
0 2,500 5,000 10,000 Feet

1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

BASELINE CONDITION
0.2% ACE FLOOD
(HEC-RAS)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend
Max Flow Depth (ft)

0.1 - 1.00
1.01 - 2.00
2.01 - 3.00
3.01 - 5.00
5.01 - 8.00
8.01 - 10.00
10.01 - 15.00
15.01 - 15.15

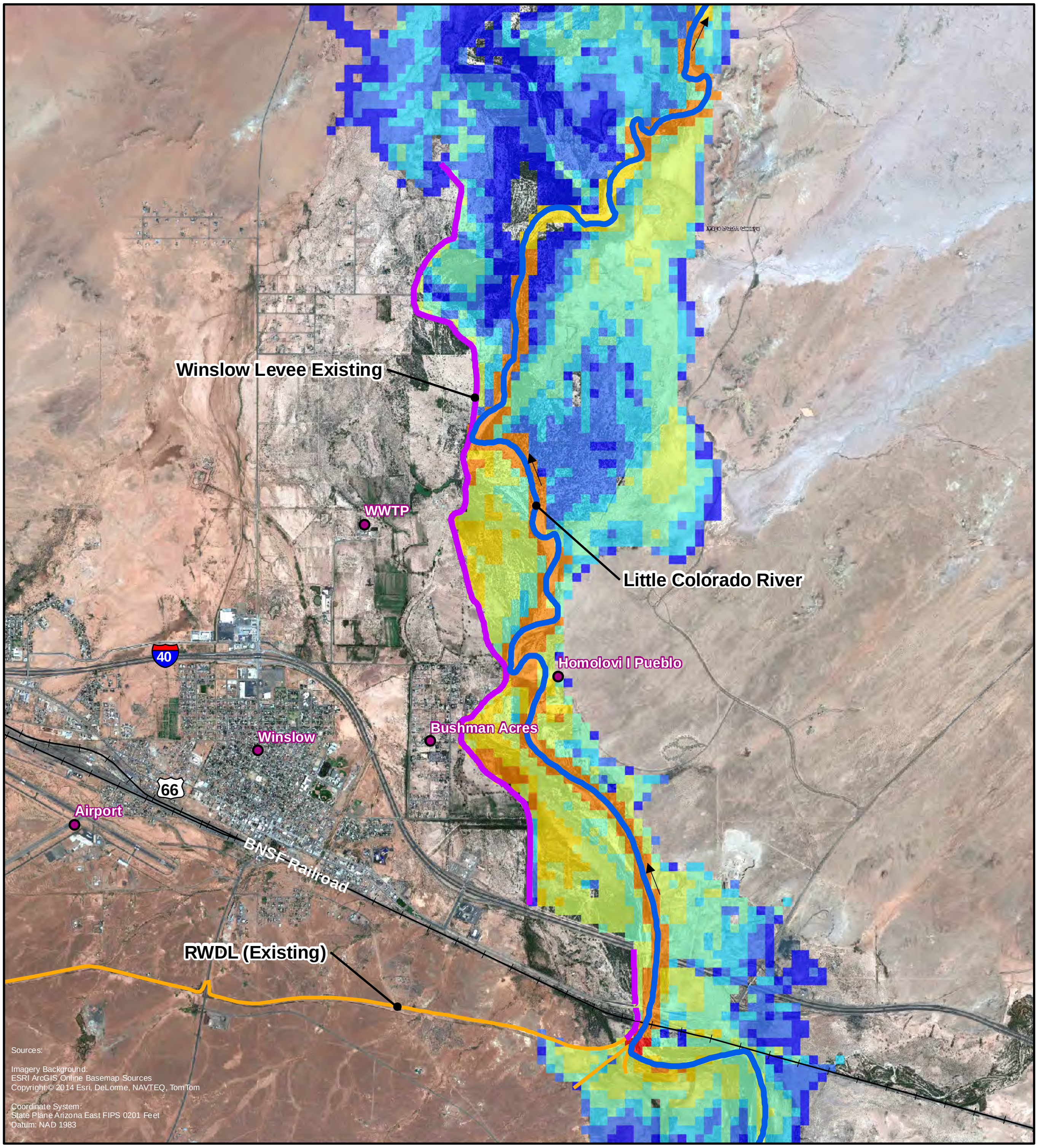
N

0 2,500 5,000 10,000 Feet
1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

BASELINE CONDITION
4% ACE FLOOD
MAX FLOW DEPTH
(FLO-2D)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

Max Flow Depth (ft)

0.1 - 1.00	5.01 - 8.00
1.01 - 2.00	8.01 - 10.00
2.01 - 3.00	10.01 - 15.00
3.01 - 5.00	15.01 - 16.52

N

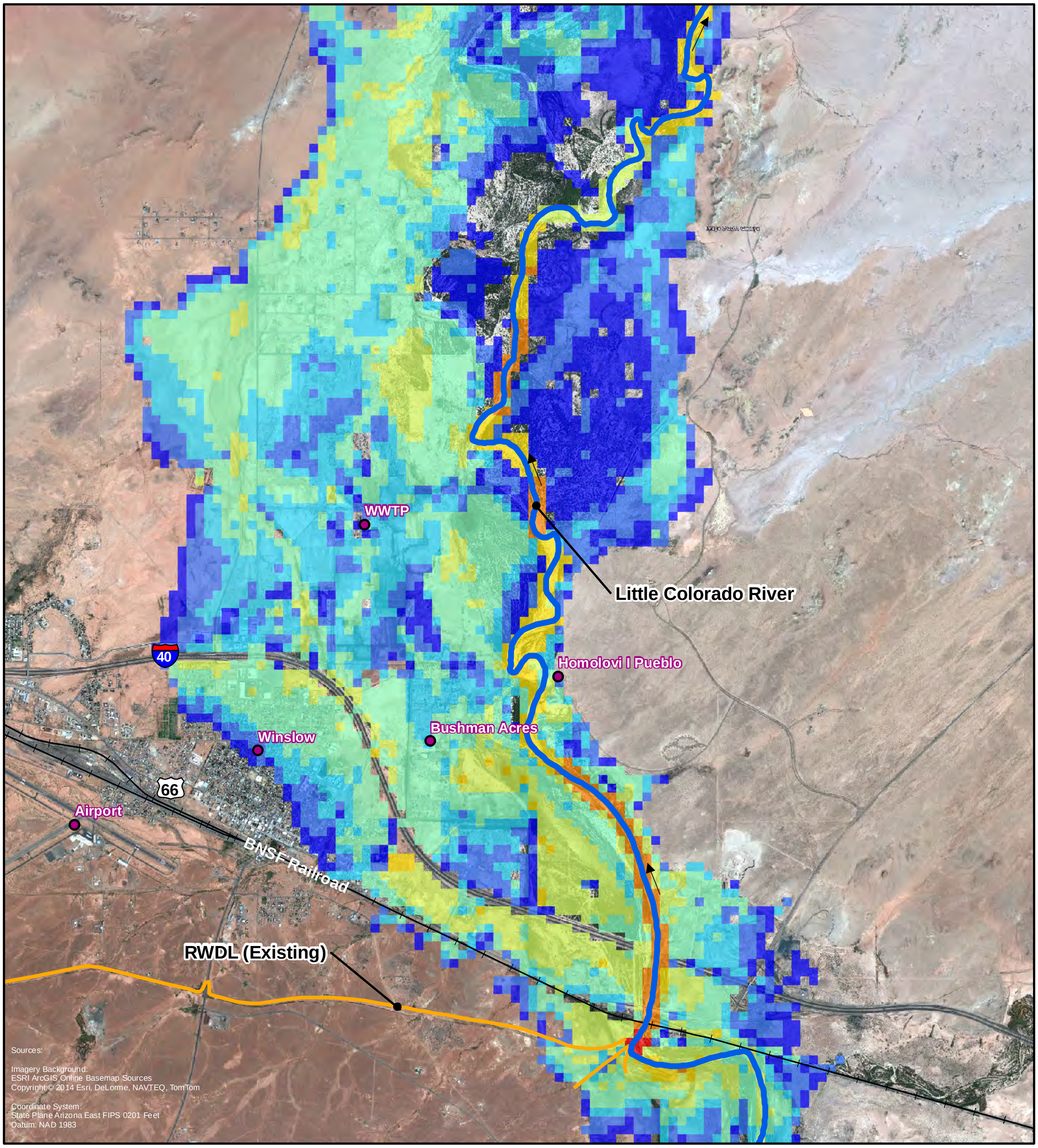
0 2,500 5,000 10,000 Feet

1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

BASELINE CONDITION
2% ACE FLOOD
MAX FLOW DEPTH
(FLO-2D)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend
Max Flow Depth (ft)

0.1 - 1.00	5.01 - 8.00
1.01 - 2.00	8.01 - 10.00
2.01 - 3.00	10.01 - 15.00
3.01 - 5.00	15.01 - 17.06

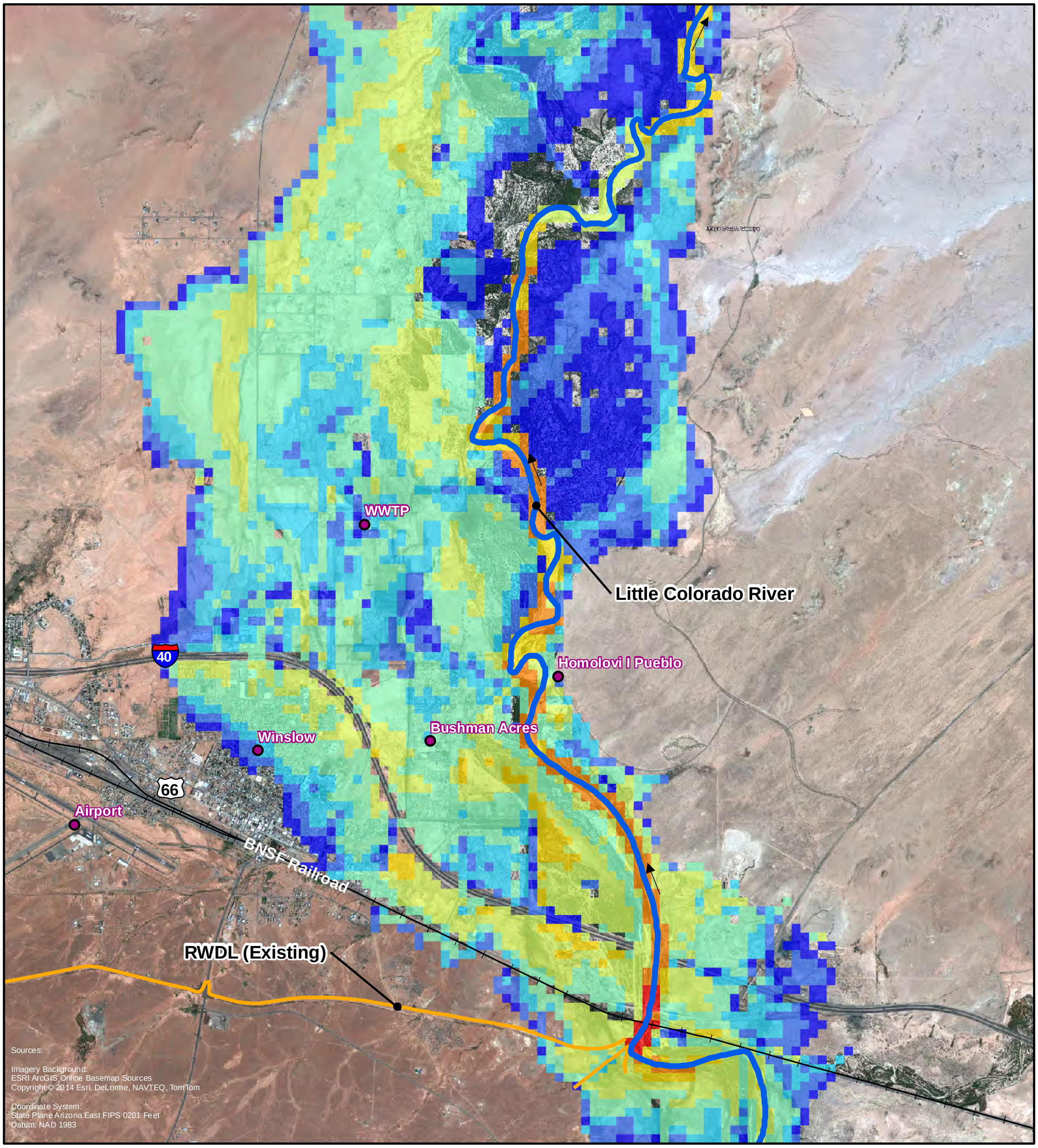
N

0 2,500 5,000 10,000 Feet
1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

BASELINE CONDITION
1% ACE FLOOD
MAX FLOW DEPTH
(FLO-2D)
NO LEVEE

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

Max Flow Depth (ft)

0.1 - 1.00	5.01 - 8.00
1.01 - 2.00	8.01 - 10.00
2.01 - 3.00	10.01 - 15.00
3.01 - 5.00	15.01 - 18.50

N

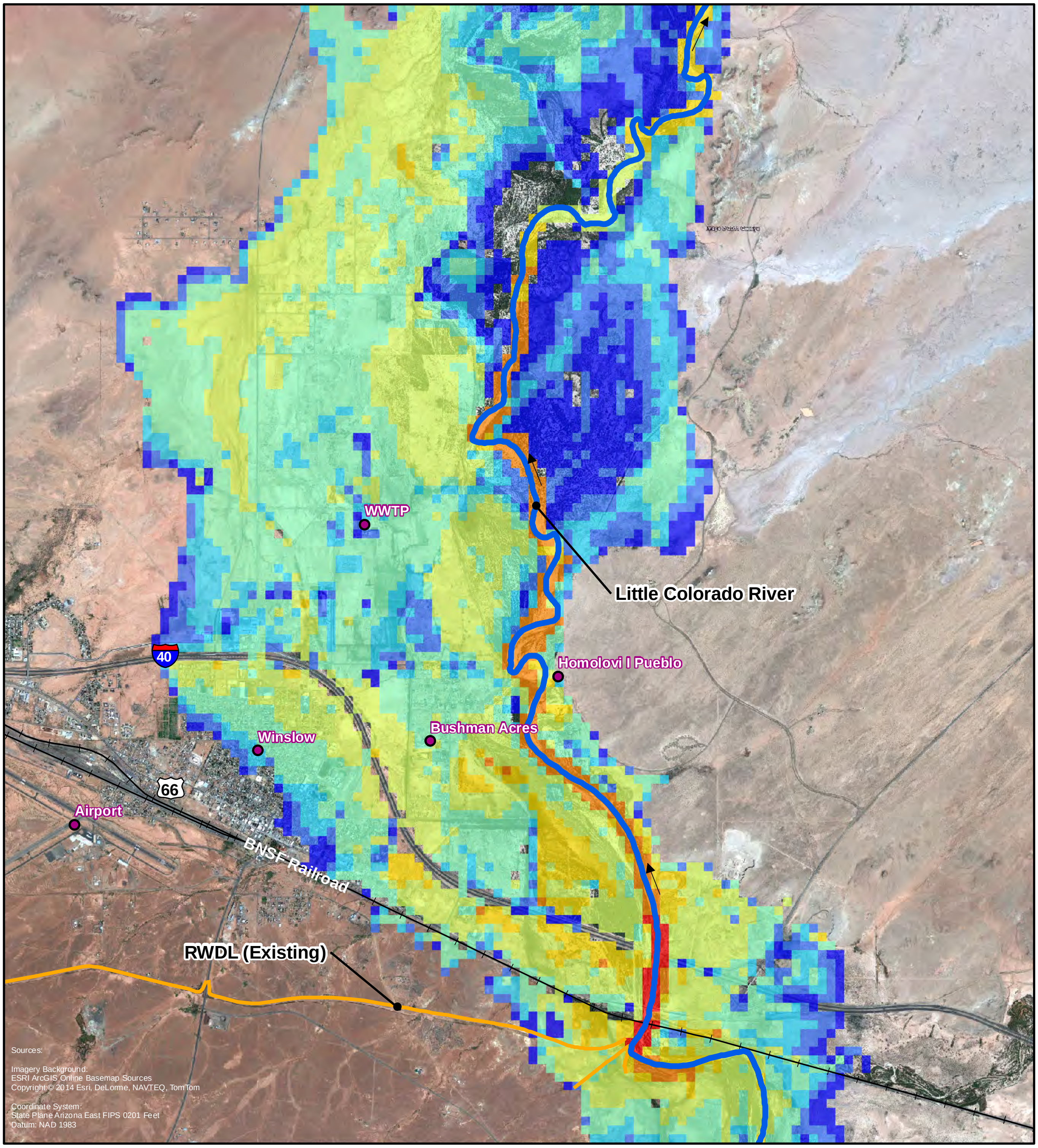
0 2,500 5,000 10,000 Feet

1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

BASELINE CONDITION
0.5% ACE FLOOD
MAX FLOW DEPTH
(FLO-2D)
NO LEVEE

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend
Max Flow Depth (ft)

- 0.1 - 1.00
- 1.01 - 2.00
- 2.01 - 3.00
- 3.01 - 5.00
- 5.01 - 8.00
- 8.01 - 10.00
- 10.01 - 15.00
- 15.01 - 20.71

N

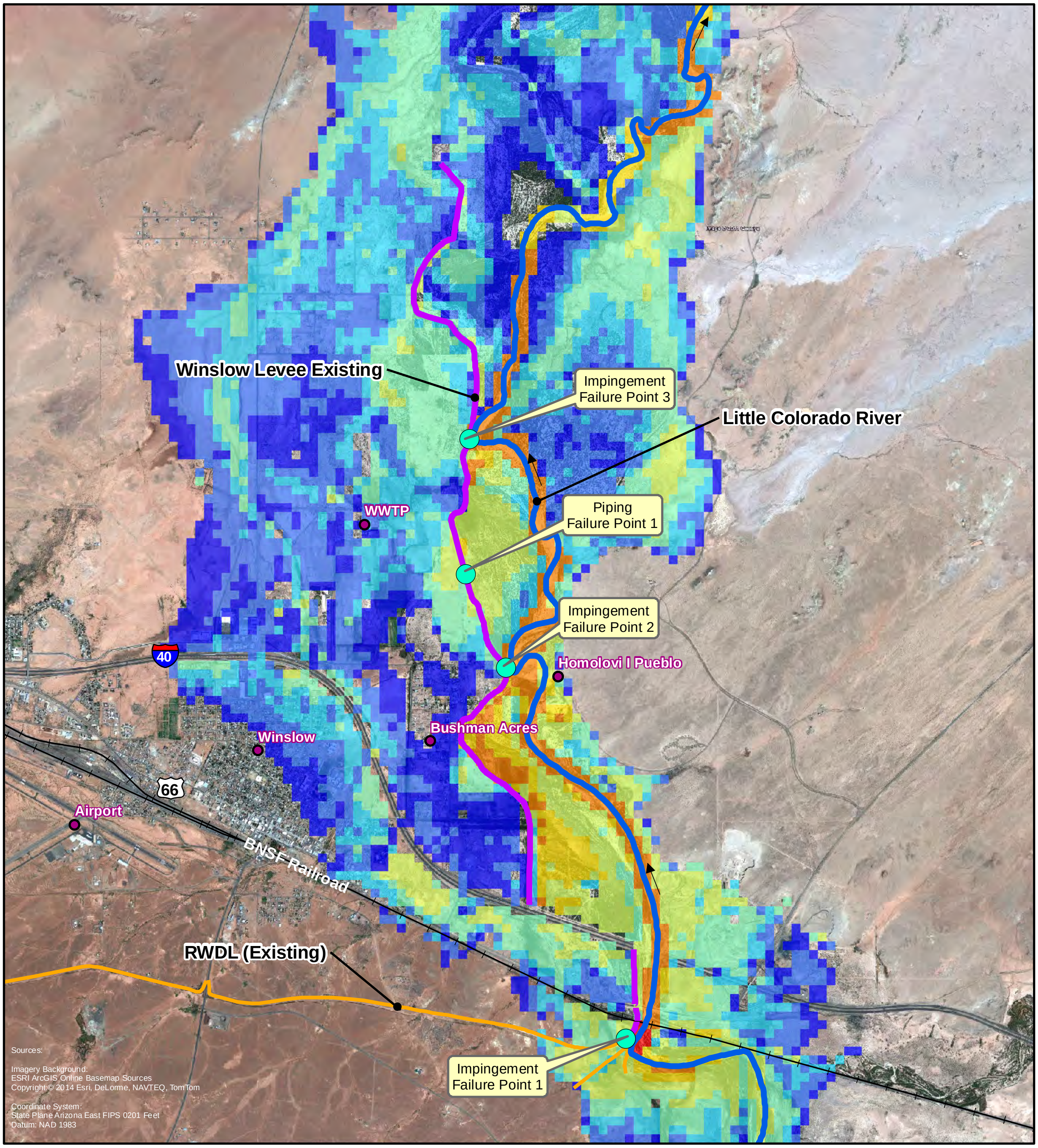
0 2,500 5,000 10,000 Feet

1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

BASELINE CONDITION
0.2% ACE FLOOD
MAX FLOW DEPTH
(FLO-2D)
NO LEVEE

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend
Max Flow Depth (ft)

0.1 - 1.00	5.01 - 8.00
1.01 - 2.00	8.01 - 10.00
2.01 - 3.00	10.01 - 15.00
3.01 - 5.00	15.01 - 17.60

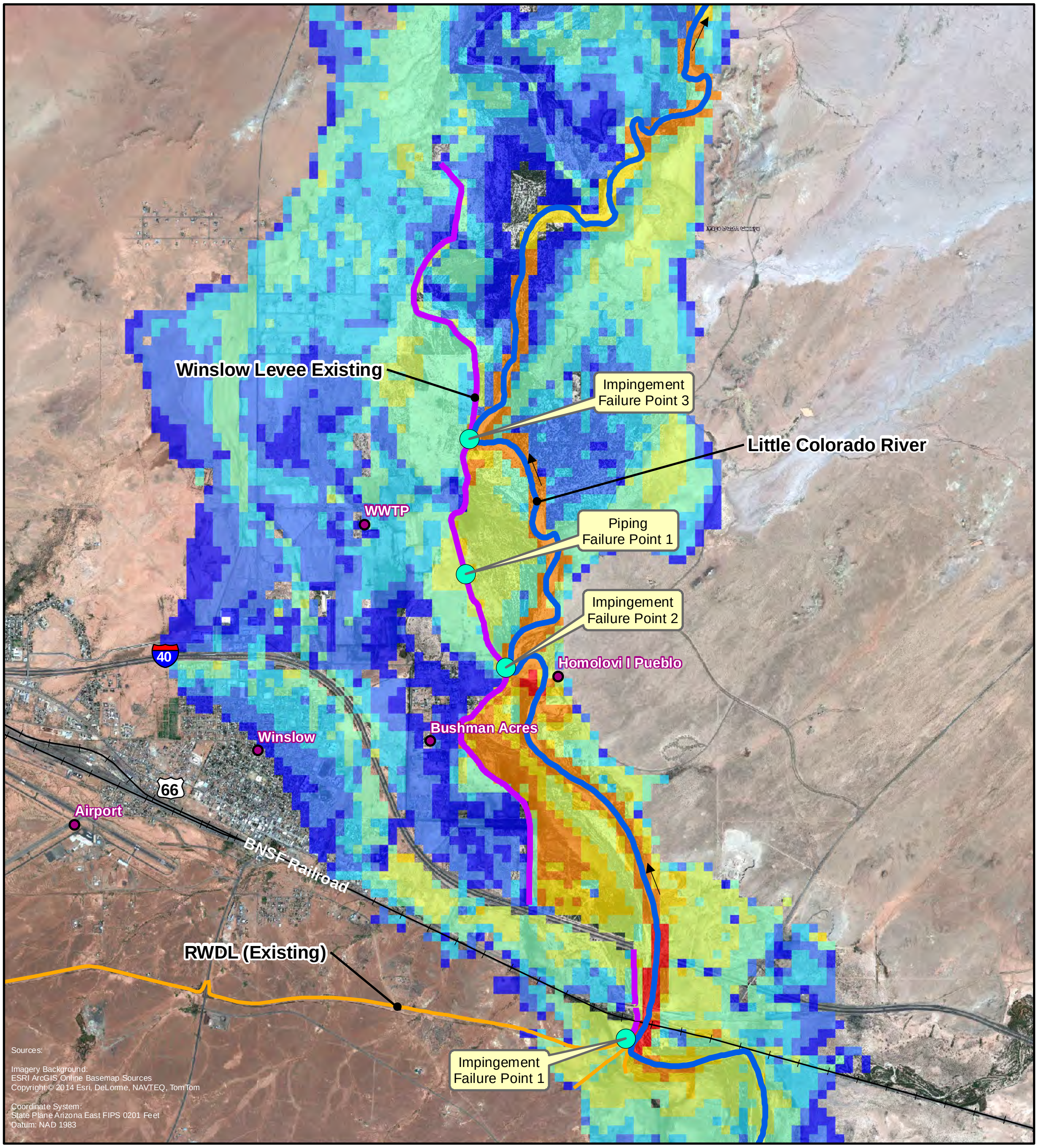
N

0 2,500 5,000 10,000 Feet
1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

1% ACE FLOOD
MAX FLOW DEPTH
IMPINGEMENT AND
PIPING FAILURE

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend
Max Flow Depth (ft)

- 0.1 - 1.00
- 1.01 - 2.00
- 2.01 - 3.00
- 3.01 - 5.00
- 5.01 - 8.00
- 8.01 - 10.00
- 10.01 - 15.00
- 15.01 - 19.09

N

0 2,500 5,000 10,000 Feet

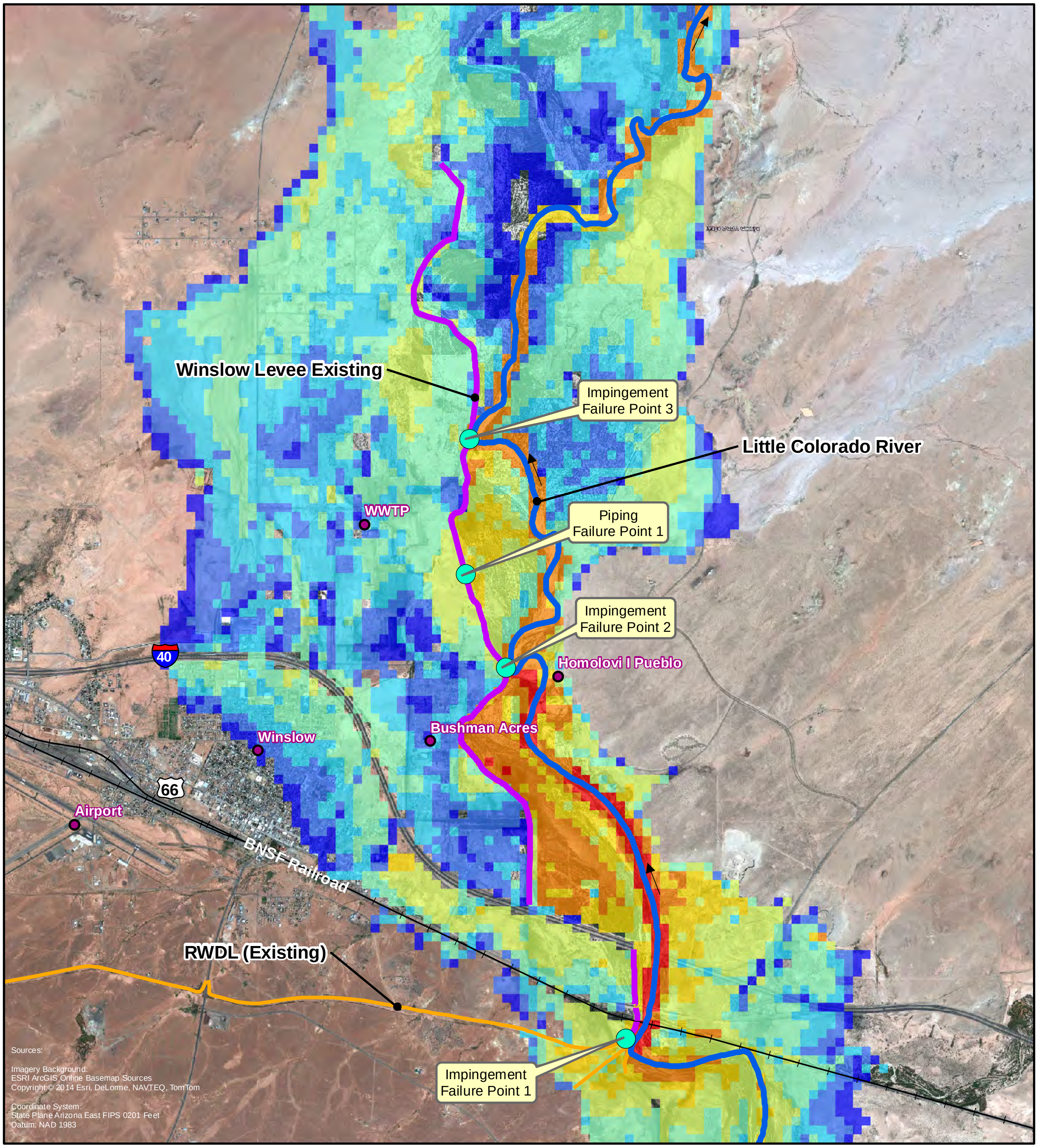
1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

0.5% ACE FLOOD
MAX FLOW DEPTH
IMPINGEMENT AND
PIPING FAILURE

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 22



Legend
Max Flow Depth (ft)

0.1 - 1.00	5.01 - 8.00
1.01 - 2.00	8.01 - 10.00
2.01 - 3.00	10.01 - 15.00
3.01 - 5.00	15.01 - 21.25

N

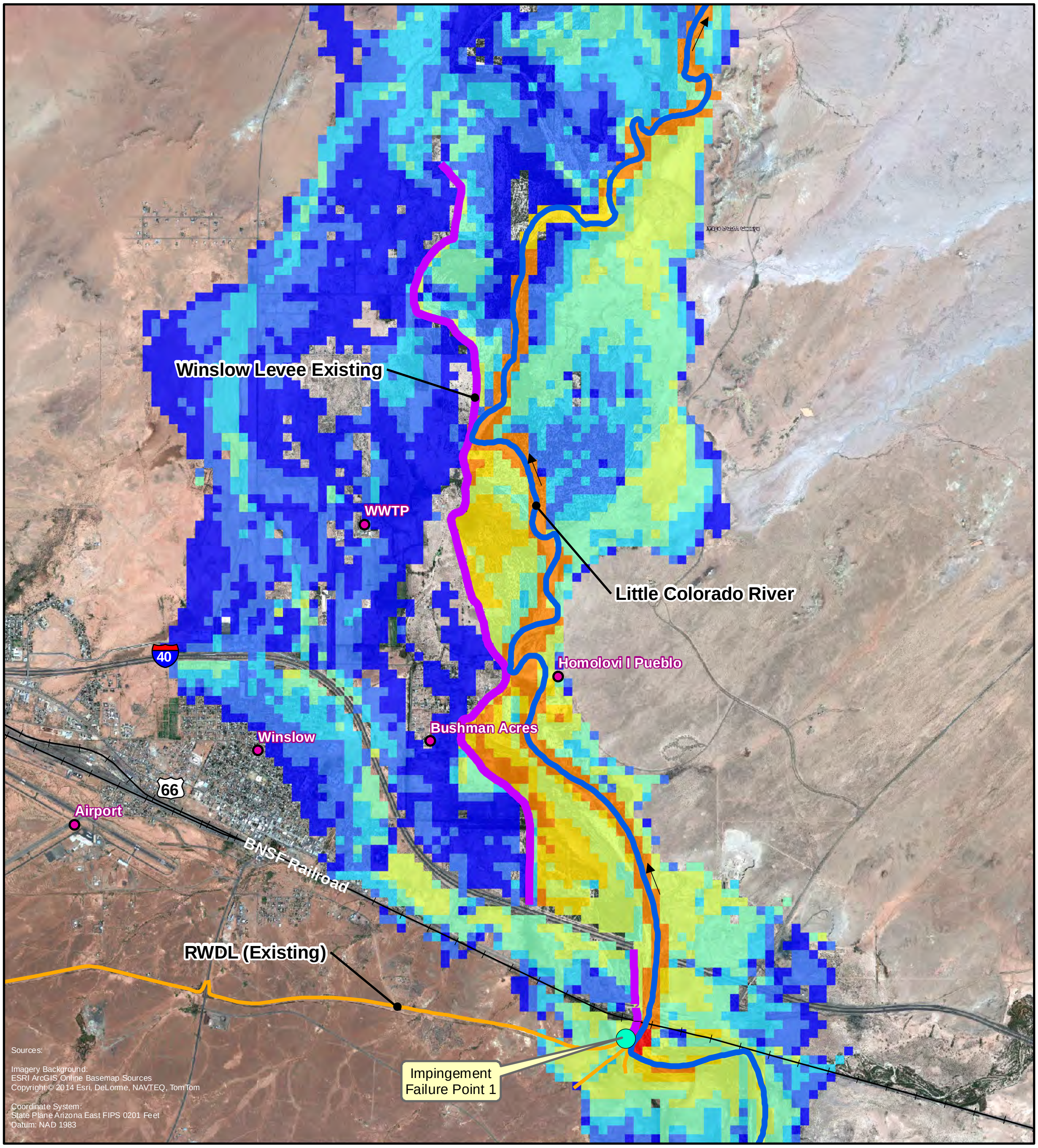
0 2,500 5,000 10,000 Feet

1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

0.2% ACE FLOOD
MAX FLOW DEPTH
IMPINGEMENT AND
PIPING FAILURE

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend
Max Flow Depth (ft)

0.1 - 1.00	5.01 - 8.00
1.01 - 2.00	8.01 - 10.00
2.01 - 3.00	10.01 - 15.00
3.01 - 5.00	15.01 - 17.60

N

0 2,500 5,000 10,000 Feet

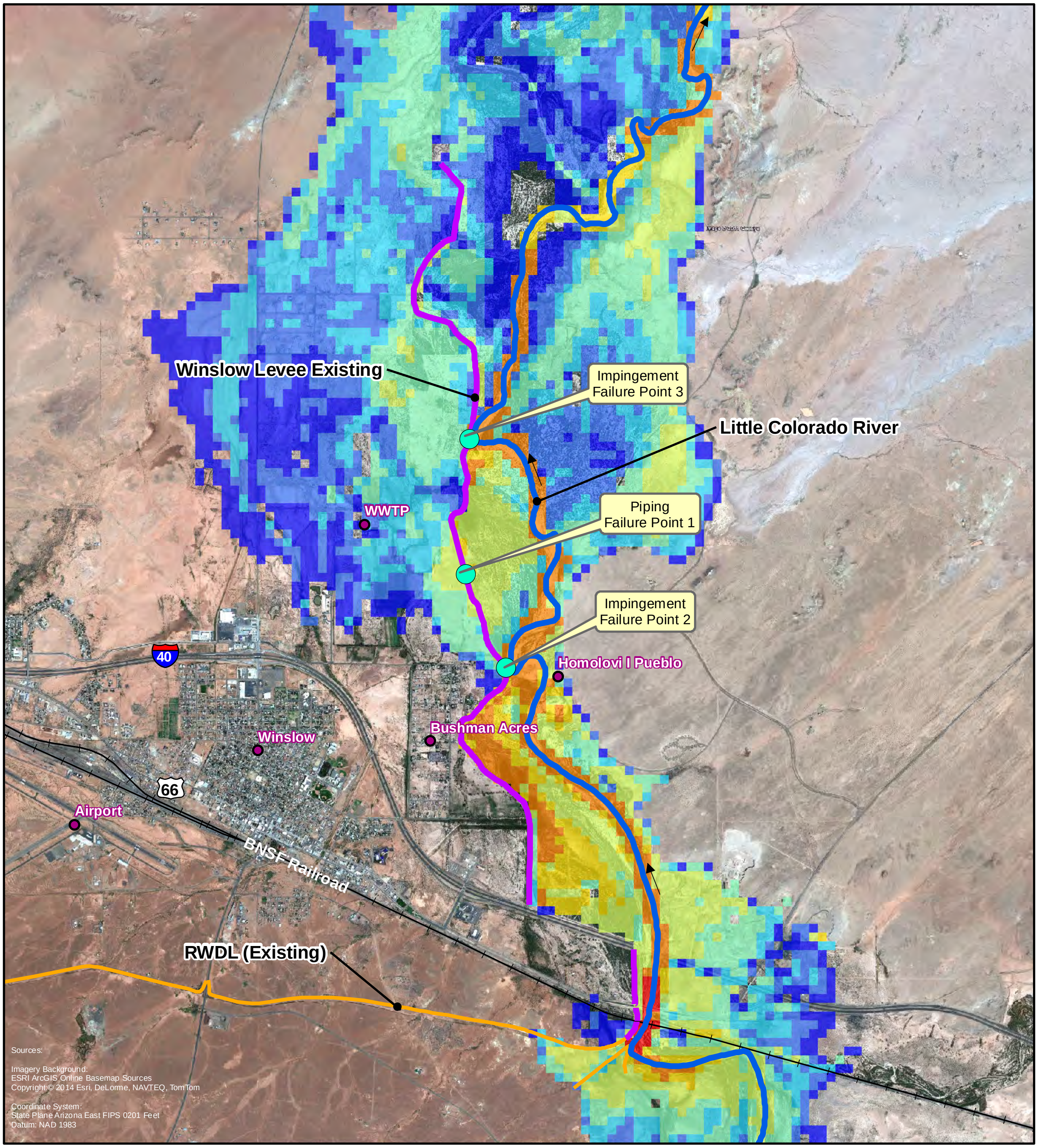
1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

1% ACE FLOOD
MAX FLOW DEPTH
LEVEE FAILURE
UPSTREAM

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 24



Legend
Max Flow Depth (ft)

0.1 - 1.00	5.01 - 8.00
1.01 - 2.00	8.01 - 10.00
2.01 - 3.00	10.01 - 15.00
3.01 - 5.00	15.01 - 17.96

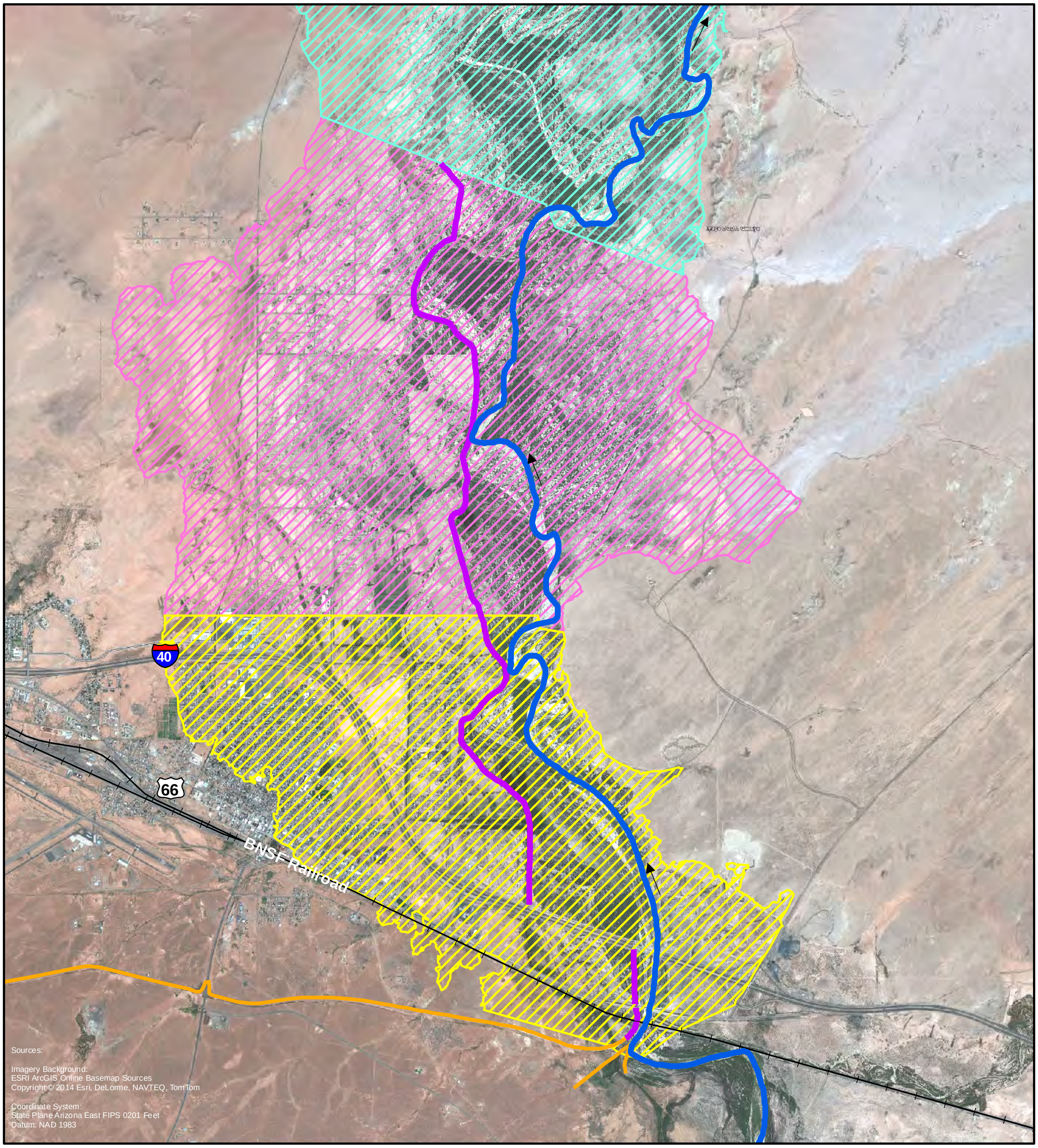
N

0 2,500 5,000 10,000 Feet
1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

1% ACE FLOOD
MAX FLOW DEPTH
LEVEE FAILURE
DOWNSTREAM

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad
- Index Reach 1
- Index Reach 2
- Index Reach 3

N

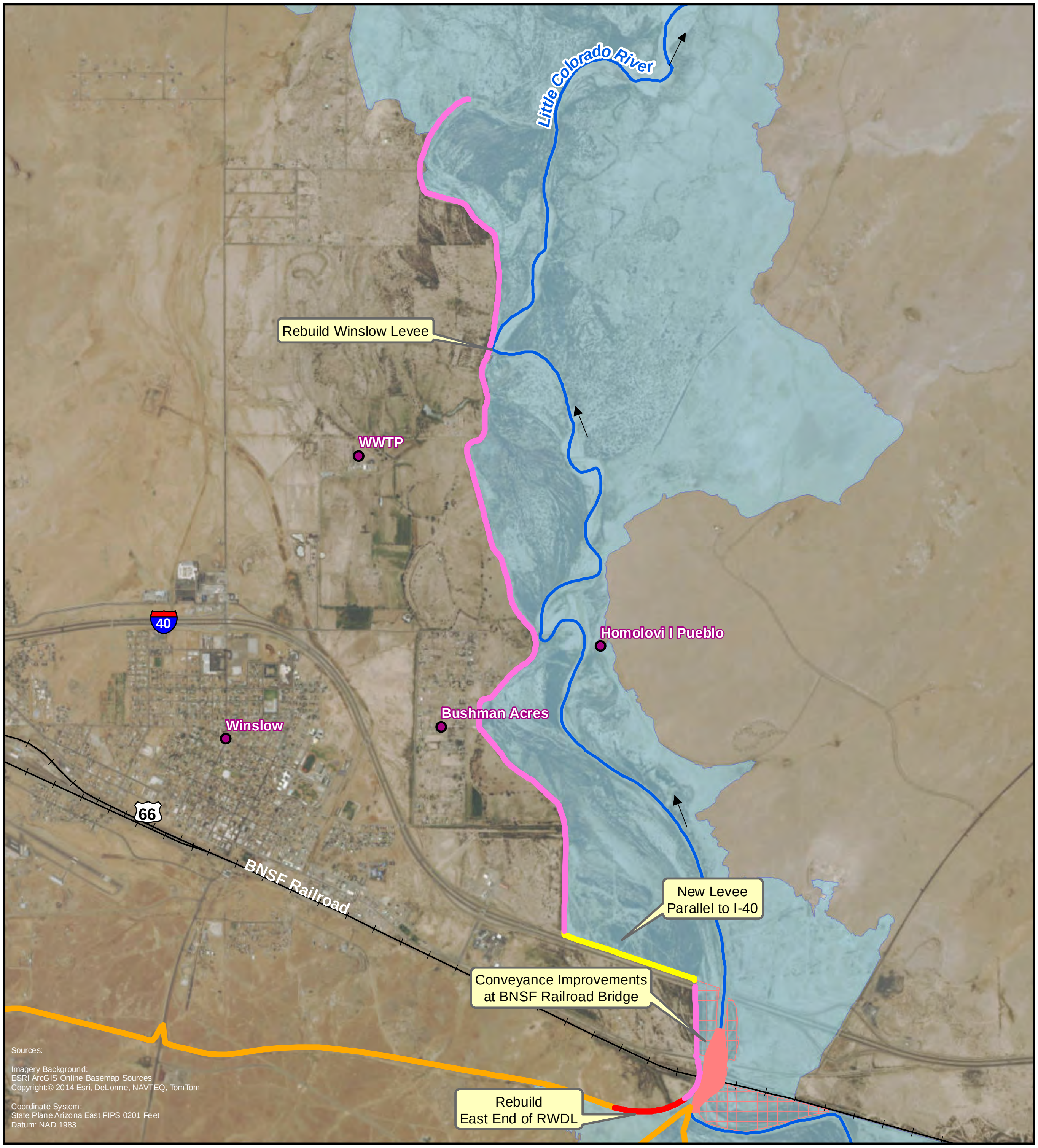
0 2,500 5,000 10,000 Feet

1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

RISK & UNCERTAINTY
INDEX REACH MAP

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Rebuild Winslow Levee
- New Levee
- Rebuild RWDL
- RWDL - No Improvements
- Little Colorado River
- BNSF Railroad
- Remove Saltcedar
- Conveyance Improvements
- 1% ACE Floodplain

N

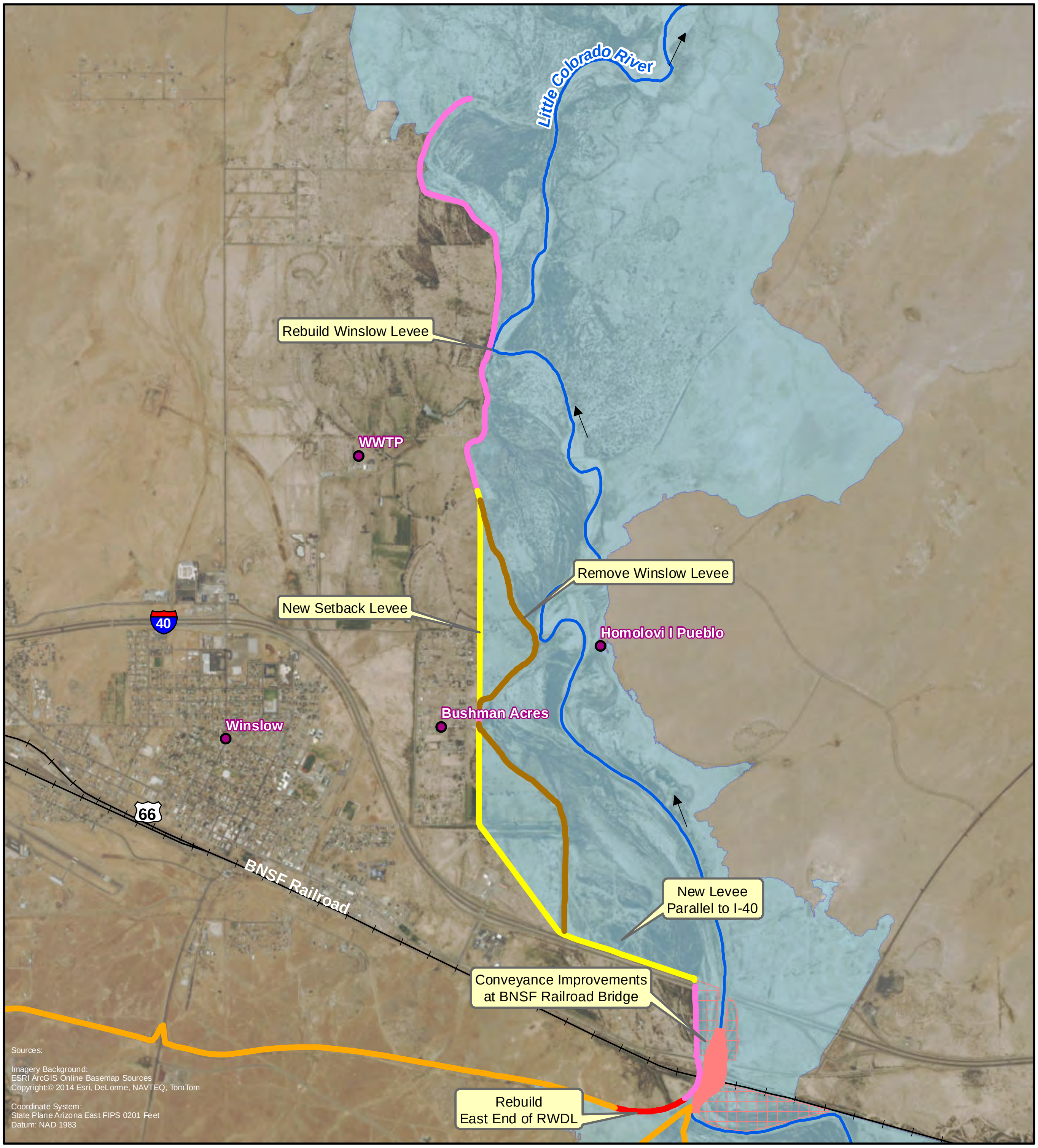
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 1.1
REBUILD LEVEES
NEW LEVEE PARALLEL TO I-40
CONVEYANCE IMPROVEMENTS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Remove Winslow Levee
- New Levee
- Rebuild Winslow Levee
- RWDL - No Improvements
- Rebuild RWDL
- Little Colorado River
- BNSF Railroad
- Conveyance Improvements
- Remove Saltcedar
- 1% ACE Floodplain

N

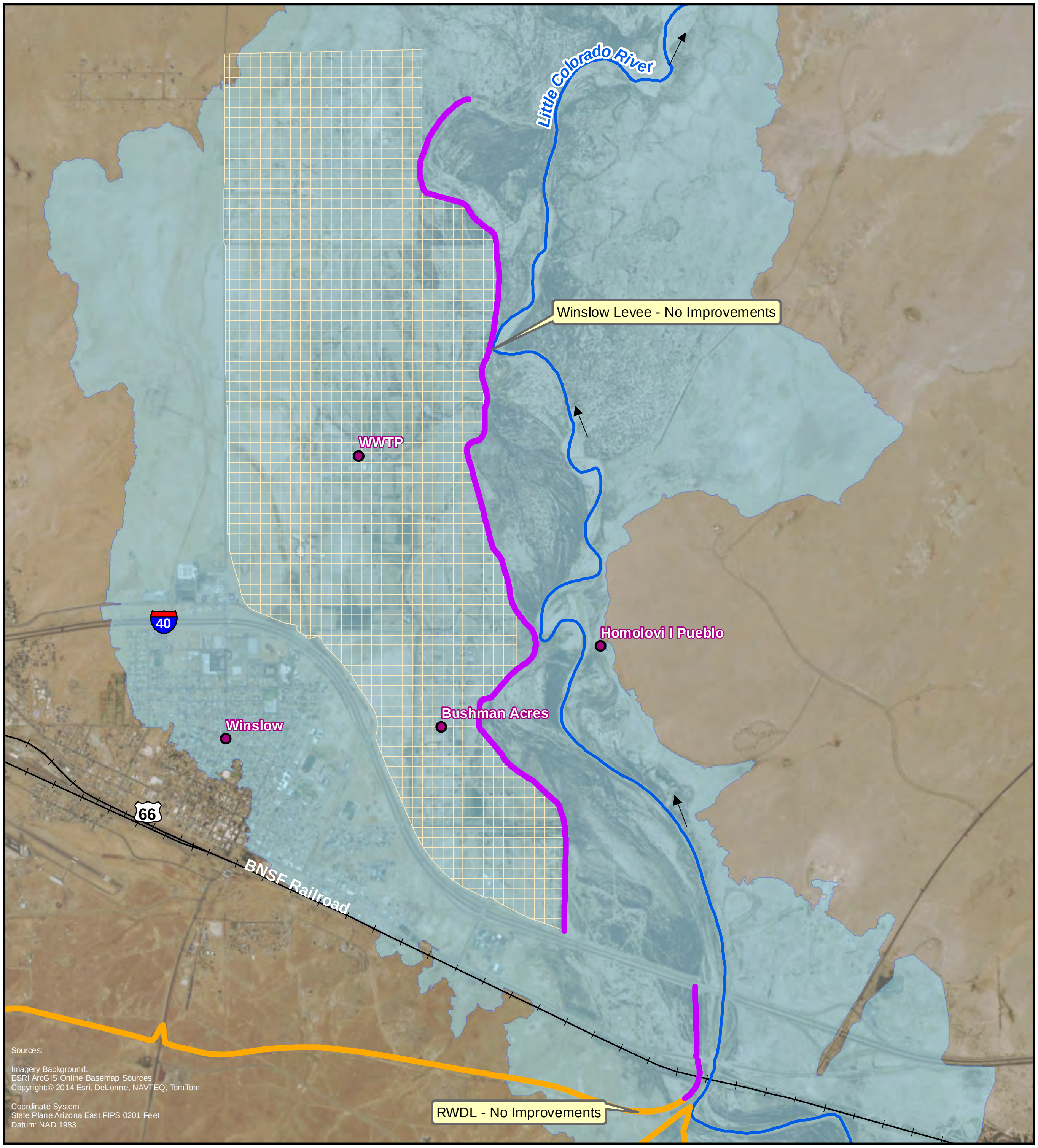
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 3.1
SETBACK LEVEE
REBUILD LEVEES
NEW LEVEE PARALLEL TO I-40
CONVEYANCE IMPROVEMENTS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

Winslow Levee - No Improvements
RWDL - No Improvements
Nonstructural Measures
Little Colorado River
BNSF Railroad
1% ACE Floodplain

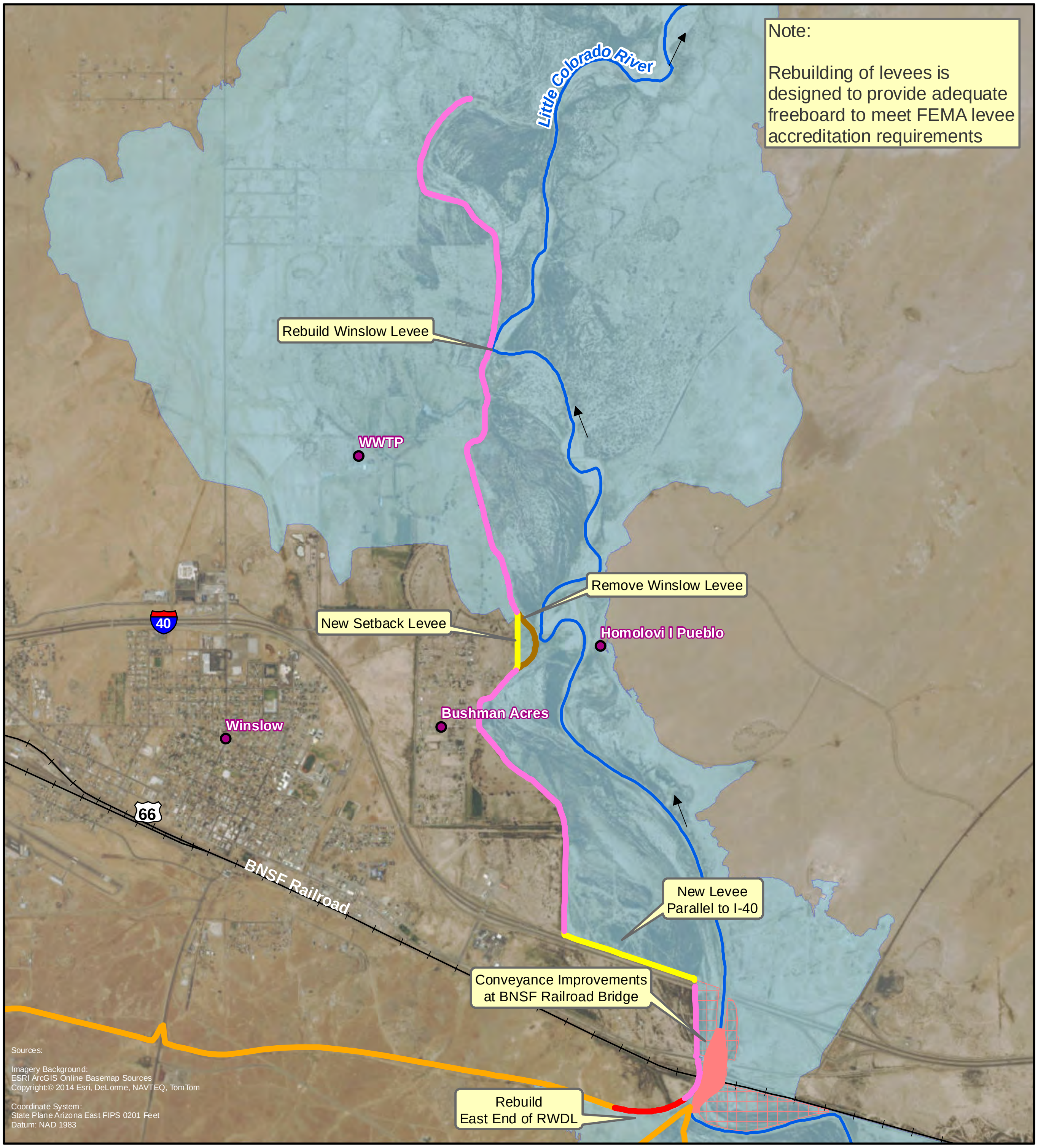
N

0 2,000 4,000 8,000 Feet
1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 7
NONSTRUCTURAL MEASURES
NORTH OF I-40
NO LEVEE OR CONVEYANCE
IMPROVEMENTS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

Remove Winslow Levee	Little Colorado River
New Levee	BNSF Railroad
Rebuild Winslow Levee	Conveyance Improvements
Rebuild RWDL	Remove Saltcedar
RWDL - No Improvements	1% ACE Floodplain

N

0 2,000 4,000 8,000 Feet

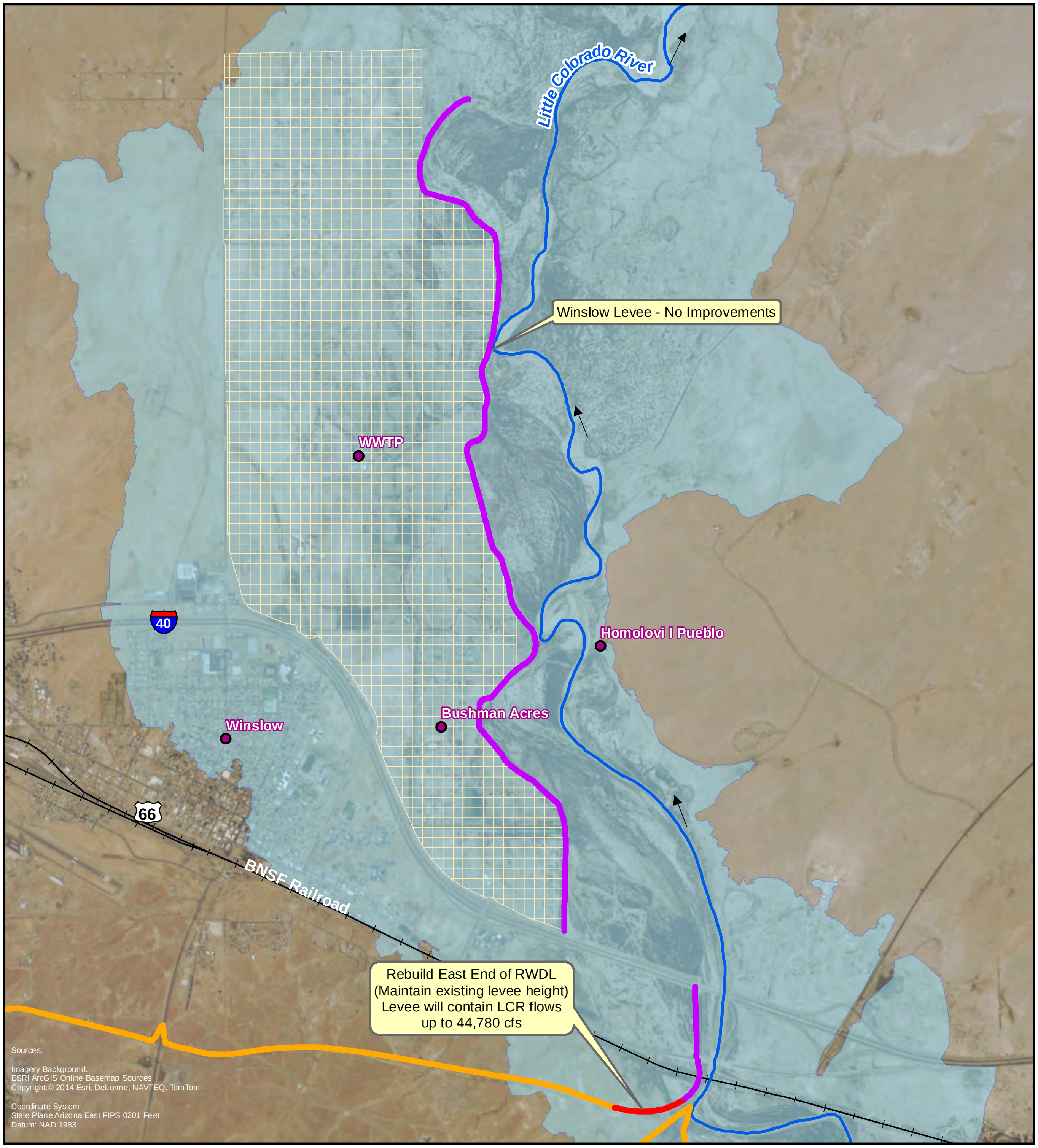
1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 8
FEMA LEVEE ACCREDITATION
REBUILD LEVEES
NEW LEVEE PARALLEL TO I-40
CONVEYANCE IMPROVEMENTS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 30



Legend

- Winslow Levee - No Improvements
- Rebuild RWDL
- RWDL - No Improvements
- Nonstructural Measures
- Little Colorado River
- BNSF Railroad
- 1% ACE Floodplain

N

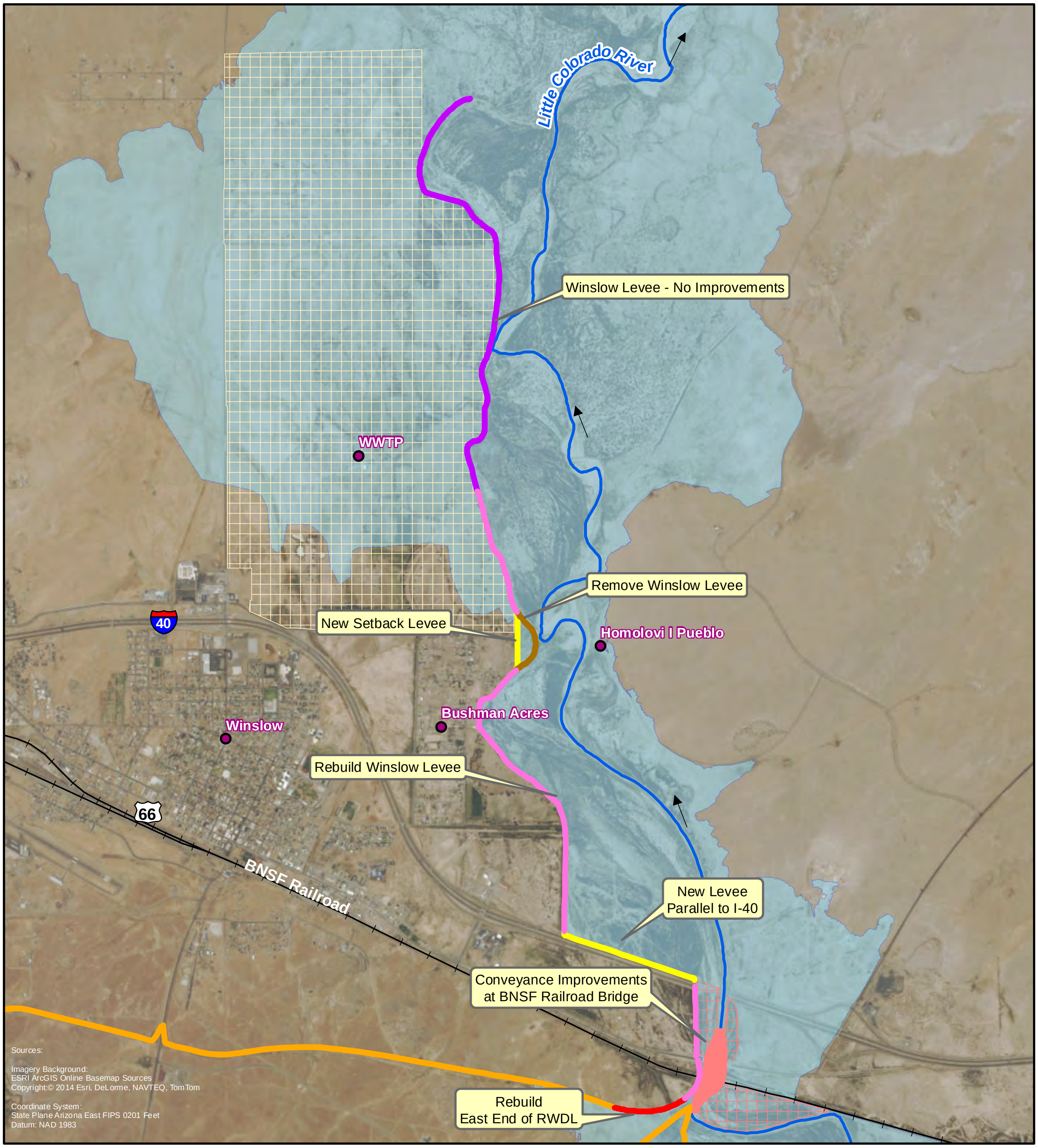
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 9
LEVEE INCREMENT 1
REBUILD RWDL
AT EXISTING HEIGHT
NO CONVEYANCE IMPROVEMENTS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Rebuild Winslow Levee
- Remove Winslow Levee
- New Levee
- Winslow Levee - No Improvements
- Rebuild RWDL
- RWDL - No Improvements
- Nonstructural Measures
- Remove Saltcedar
- Conveyance Improvements
- Little Colorado River
- BNSF Railroad
- 1% ACE Floodplain

N

0 2,000 4,000 8,000 Feet

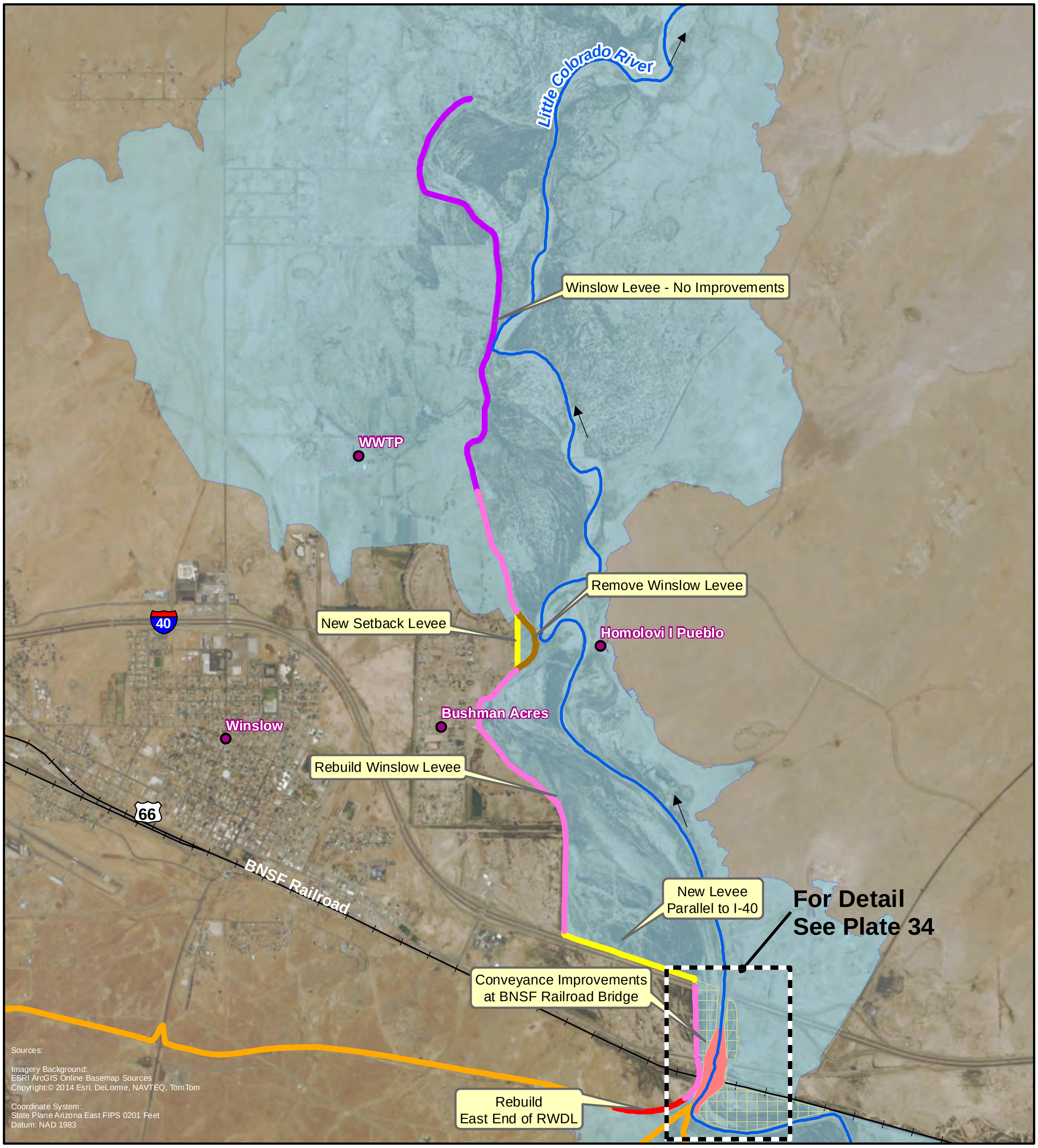
1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 10
LEVEE INCREMENTS 1 & 2
REBUILD & SETBACK LEVEES
CONVEYANCE IMPROVEMENTS
NONSTRUCTURAL MEASURES
NORTH OF I-40

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 32



Legend

- Little Colorado River
- Rebuild Winslow Levee
- Remove Winslow Levee
- New Levee
- Winslow Levee - No Improvements
- Rebuild RWDL
- RWDL - No Improvements
- Remove Saltcedar
- Conveyance Improvements
- BNSF Railroad
- 1% ACE Floodplain

N

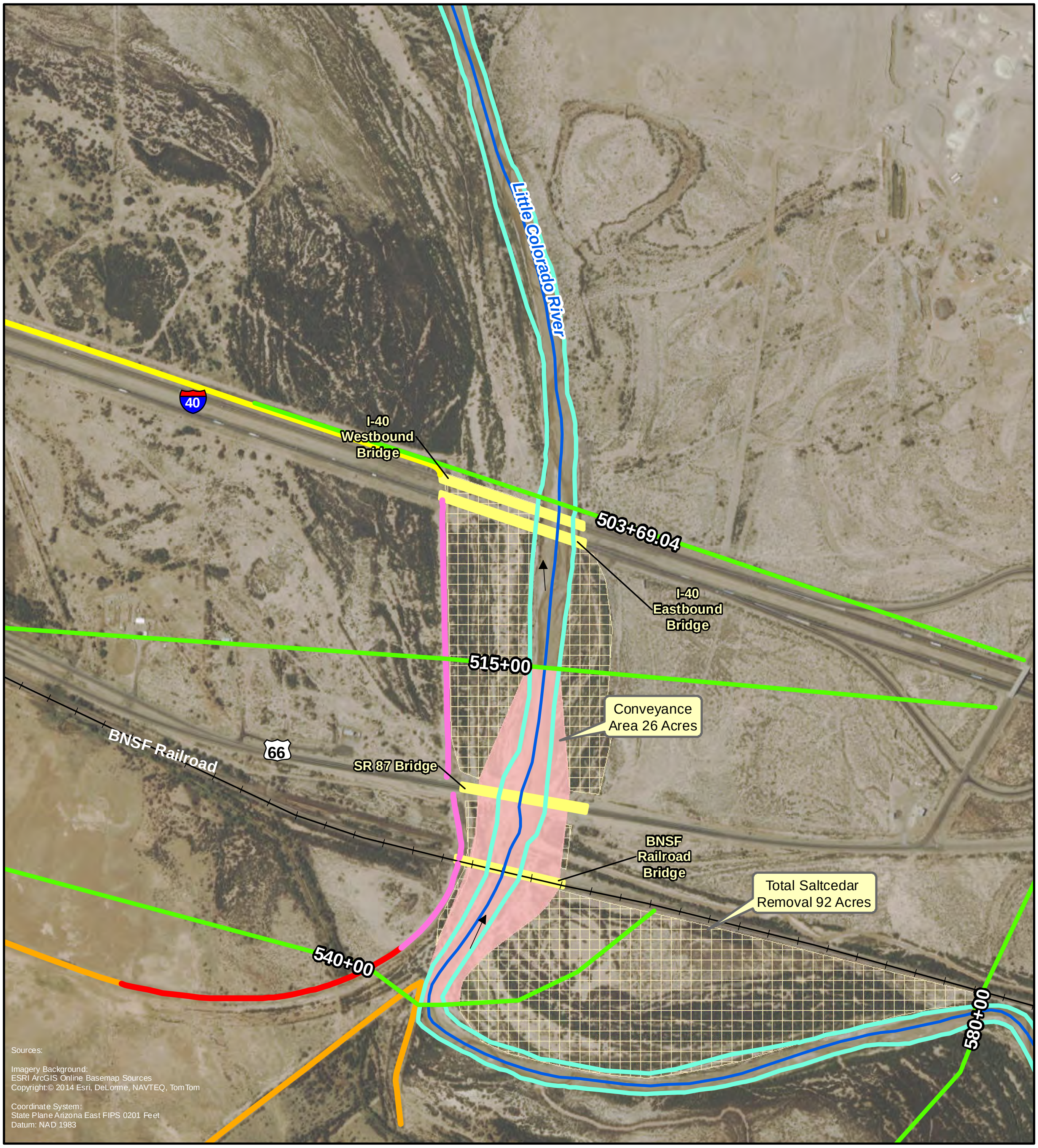
0 2,000 4,000 8,000 Feet

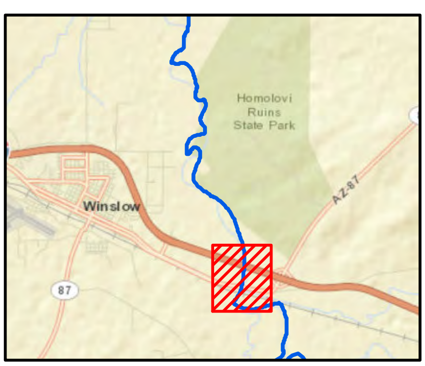
1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 10.1
REBUILD EAST END OF RWDL
REBUILD & SETBACK LEVEES
NEW LEVEE ALONG I-40
CONVEYANCE IMPROVEMENTS
1% ACE FLOOD LEVEE HEIGHTS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT





Legend

- Bounding Cross-Sections
- Little Colorado River
- Banks (Existing)
- New Levee
- Rebuild Winslow Levee
- Rebuild RWDL
- RWDL - No Improvements
- BNSF Railroad
- Bridges
- Conveyance Improvements
- Remove Saltcedar

N

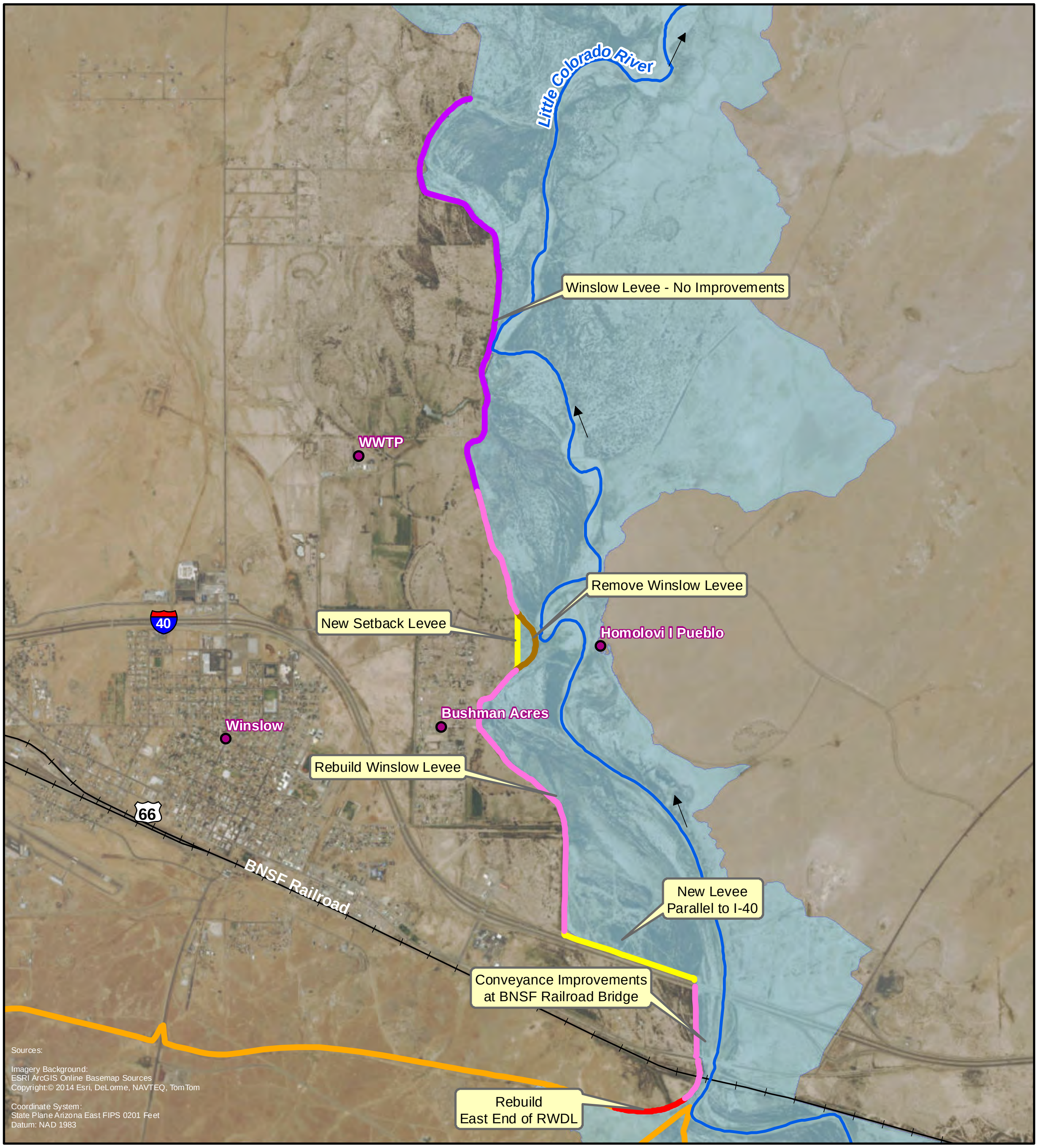
0 500 1,000 2,000 Feet

1 in = 1,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 10.1
EXCAVATE & WIDEN CHANNEL
STATION 540+00 TO 515+00
1% ACE FLOOD LEVEE HEIGHTS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Rebuild Winslow Levee
- Remove Winslow Levee
- New Levee
- Winslow Levee - No Improvements
- Rebuild RWDL
- RWDL - No Improvements
- BNSF Railroad
- 4% ACE Floodplain

N

0 2,000 4,000 8,000 Feet

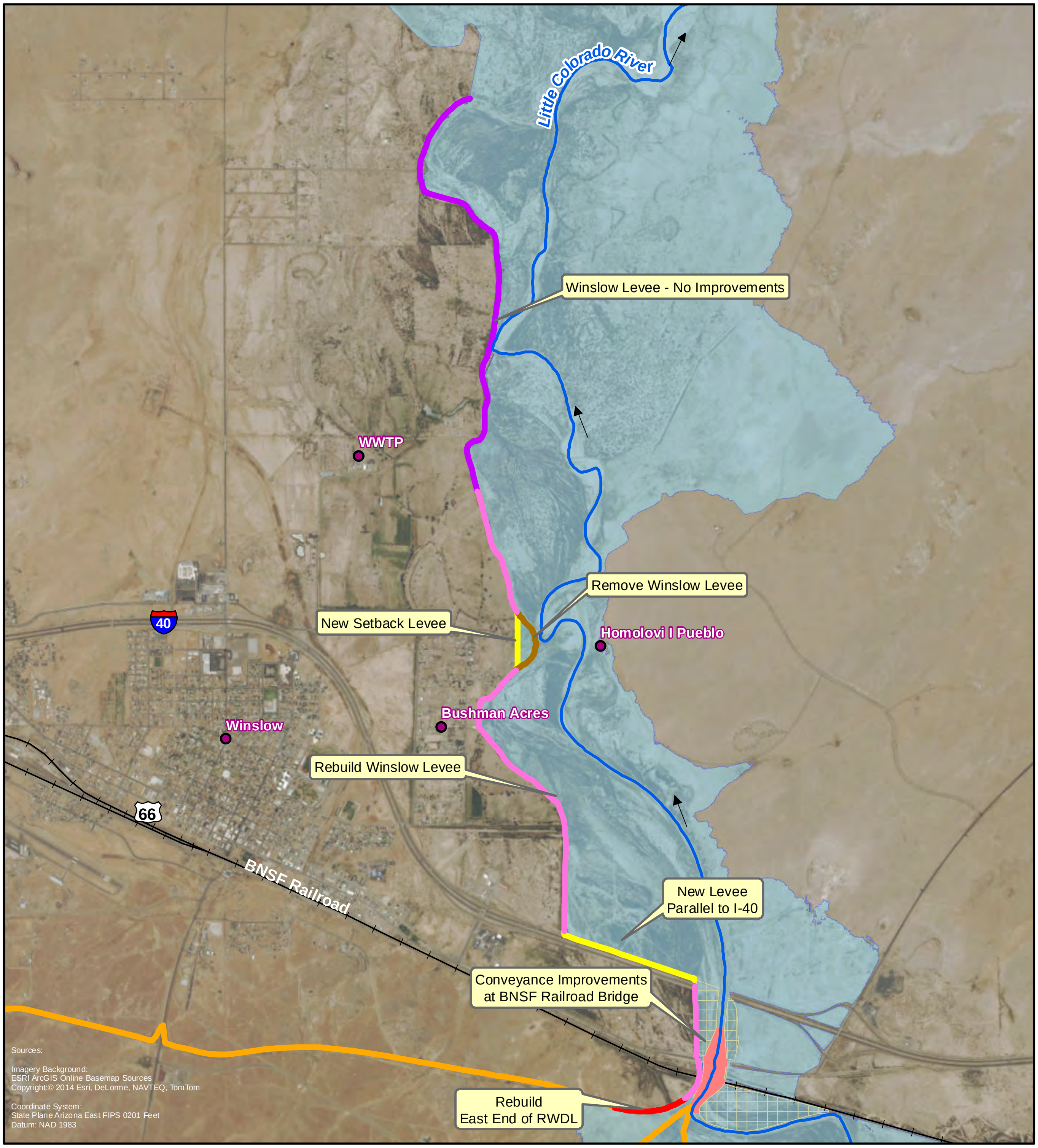
1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 10.2
REBUILD EAST END OF RWDL
REBUILD & SETBACK LEVEES
NEW LEVEE ALONG I-40
NO CONVEYANCE IMPROVEMENTS
4% ACE FLOOD LEVEE HEIGHTS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 35



Legend

- Little Colorado River
- Rebuild Winslow Levee
- Remove Winslow Levee
- New Levee
- Winslow Levee - No Improvements
- Rebuild RWDL
- RWDL - No Improvements
- Remove Saltcedar
- Conveyance Improvements
- BNSF Railroad
- 2% ACE Floodplain

N

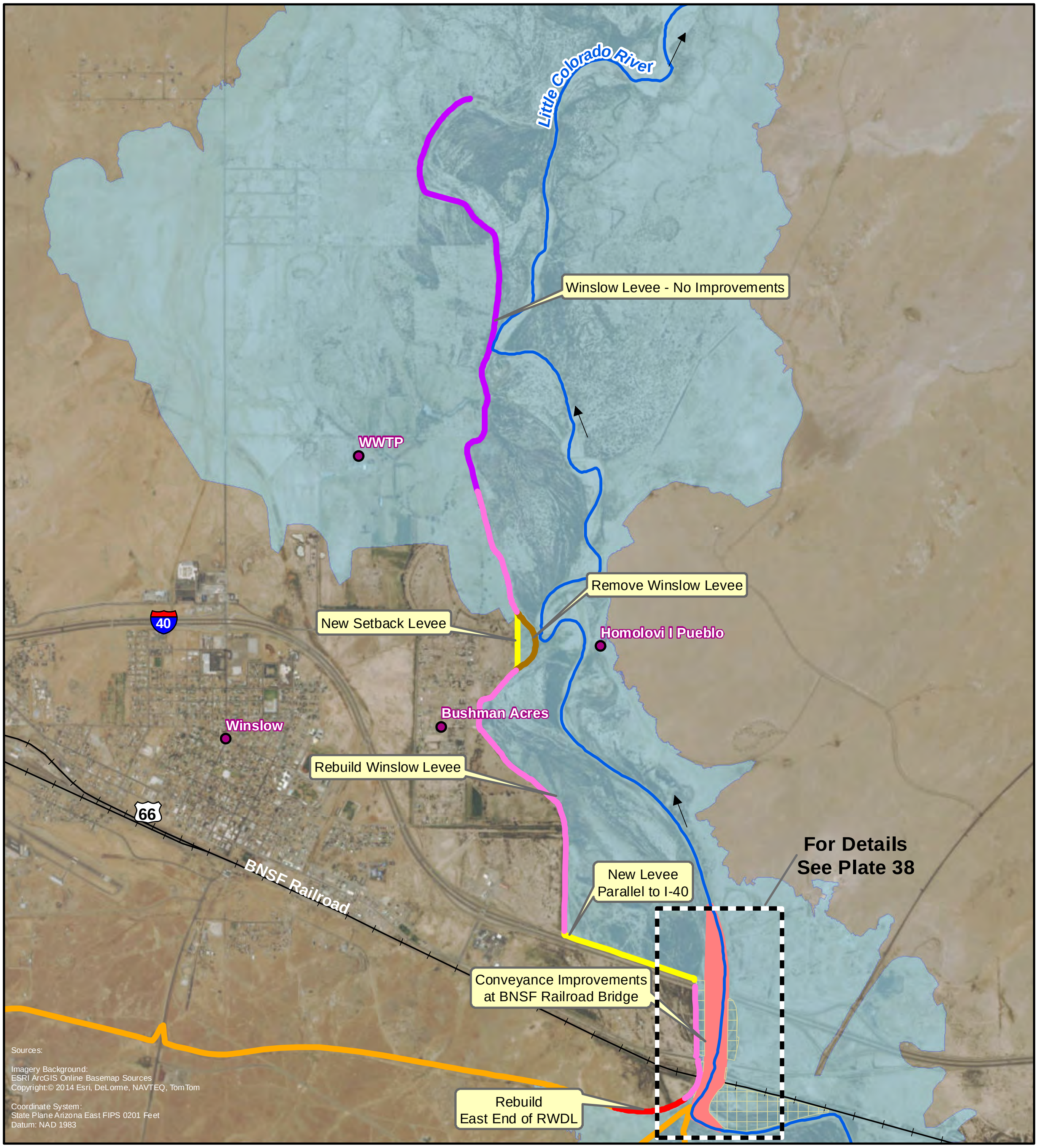
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 10.3
REBUILD EAST END OF RWDL
REBUILD & SETBACK LEVEES
NEW LEVEE ALONG I-40
CONVEYANCE IMPROVEMENTS
2% ACE FLOOD LEVEE HEIGHTS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Rebuild Winslow Levee
- Remove Winslow Levee
- New Levee
- Winslow Levee - No Improvements
- Rebuild RWDL
- RWDL - No Improvements
- Remove Saltcedar
- Conveyance Improvements
- BNSF Railroad
- 0.5-% ACE Floodplain

N

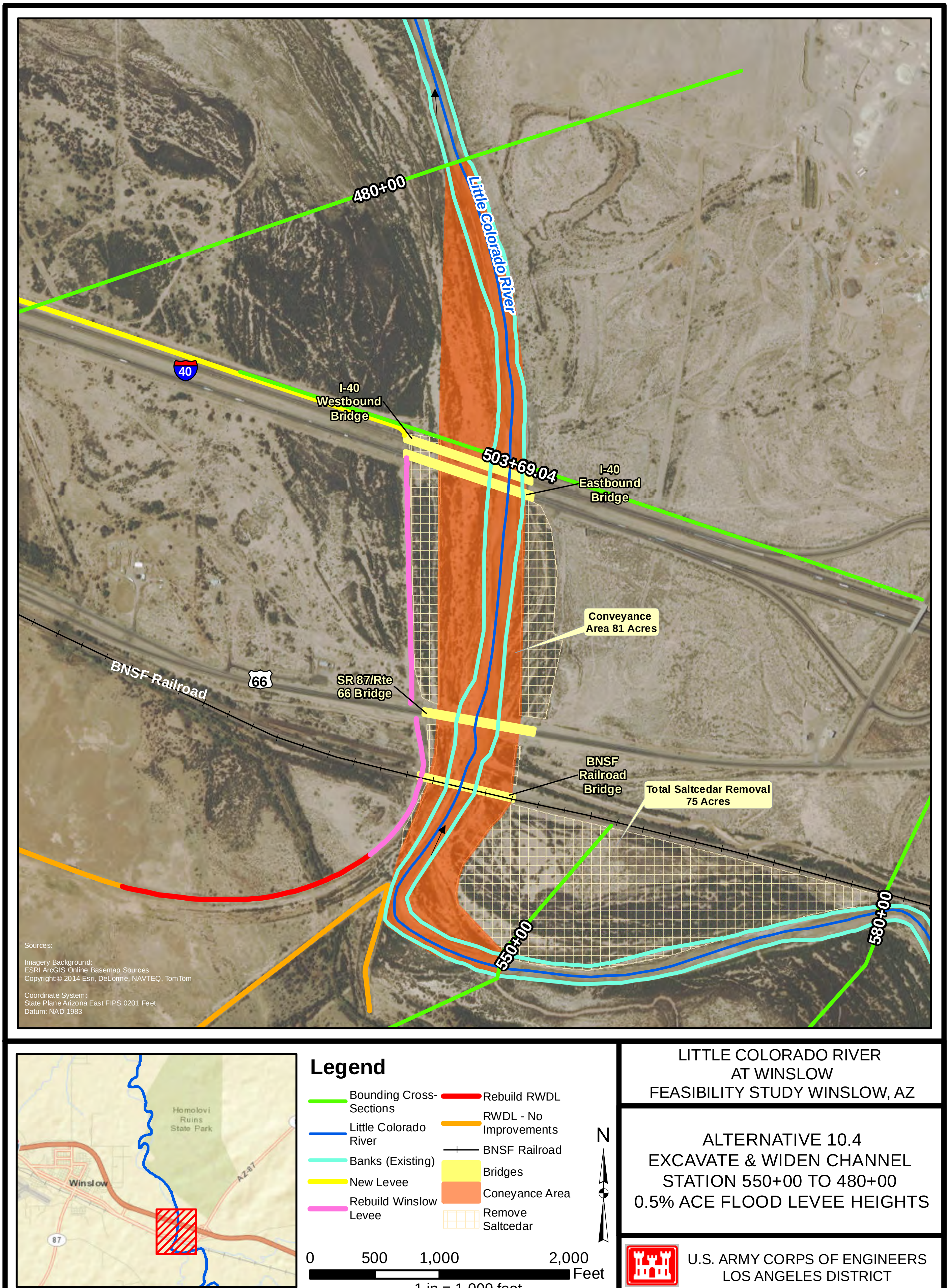
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 10.4
REBUILD EAST END OF RWDL
REBUILD & SETBACK LEVEES
NEW LEVEE ALONG I-40
CONVEYANCE IMPROVEMENTS
0.5% ACE FLOOD LEVEE HEIGHTS

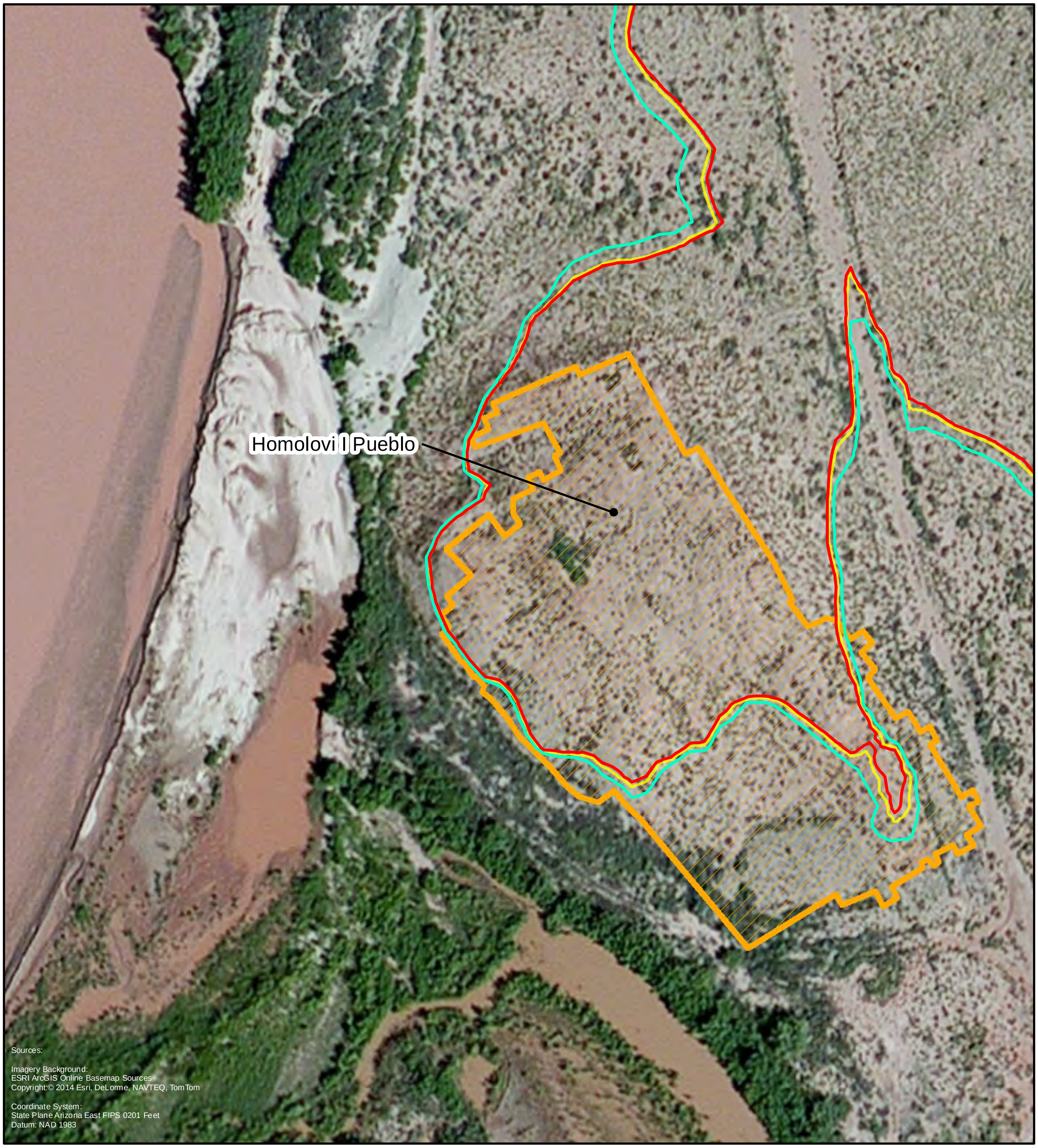
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 10.4
EXCAVATE & WIDEN CHANNEL
STATION 550+00 TO 480+00
0.5% ACE FLOOD LEVEE HEIGHTS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Homolovi I Pueblo Footprint
- Baseline- 1% ACE Floodplain
- Alternative 3.1 - 1% ACE Floodplain
- Alternative 8 & 10 - 1% ACE Floodplain

N

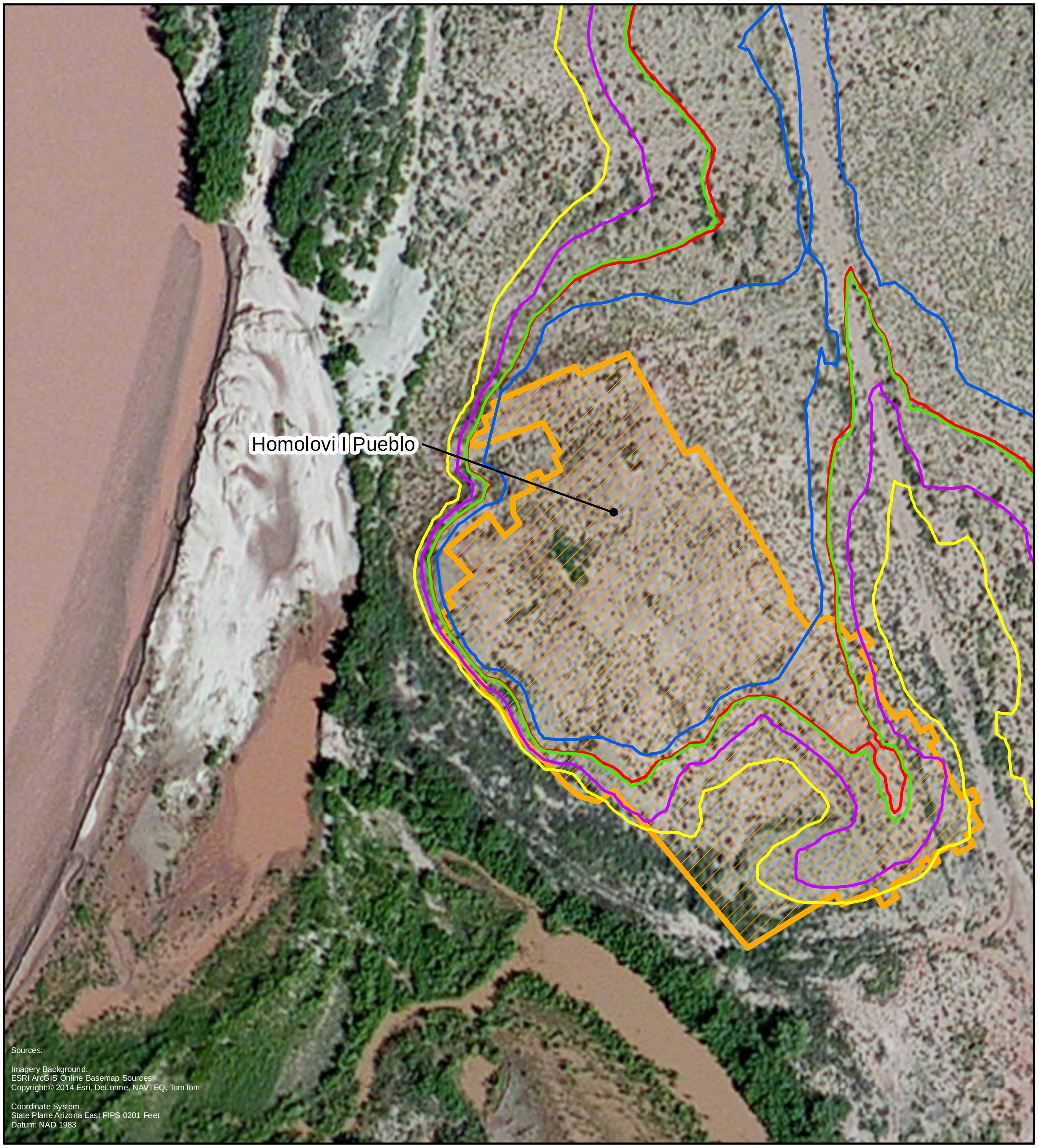
0 50 100 200 Feet

1 in = 100 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

COMPARISON OF STUDY
ALTERNATIVES TO BASELINE
HOMOLOVI I PUEBLO
1% ACE FLOOD

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Alternative 10.1 - 1% ACE Floodplain
- Alternative 10.2 - 4% ACE Floodplain
- Alternative 10.3 - 2% ACE Floodplain
- Alternative 10.4 - 0.5% ACE Floodplain
- Homolovi I Pueblo Footprint
- Baseline - 1% ACE Floodplain

N

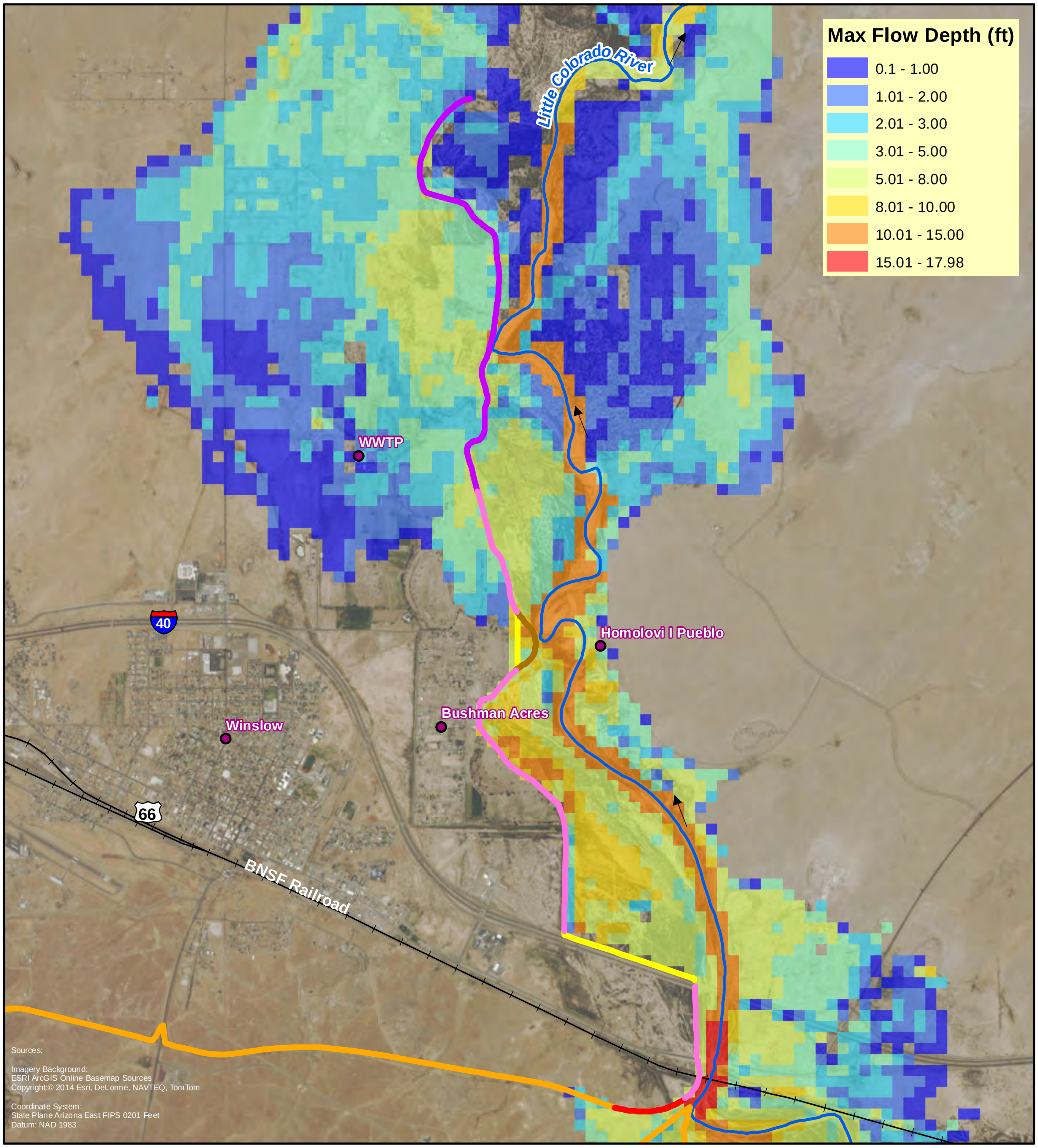
0 50 100 200 Feet

1 in = 100 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

COMPARISON OF STUDY
ALTERNATIVES TO BASELINE
HOMOLOVI I PUEBLO
1% ACE FLOOD

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Rebuild Winslow Levee
- Remove Winslow Levee
- New Levee
- Winslow Levee - No Improvements
- Rebuild RWDL
- RWDL - No Improvements
- Little Colorado River
- BNSF Railroad

N

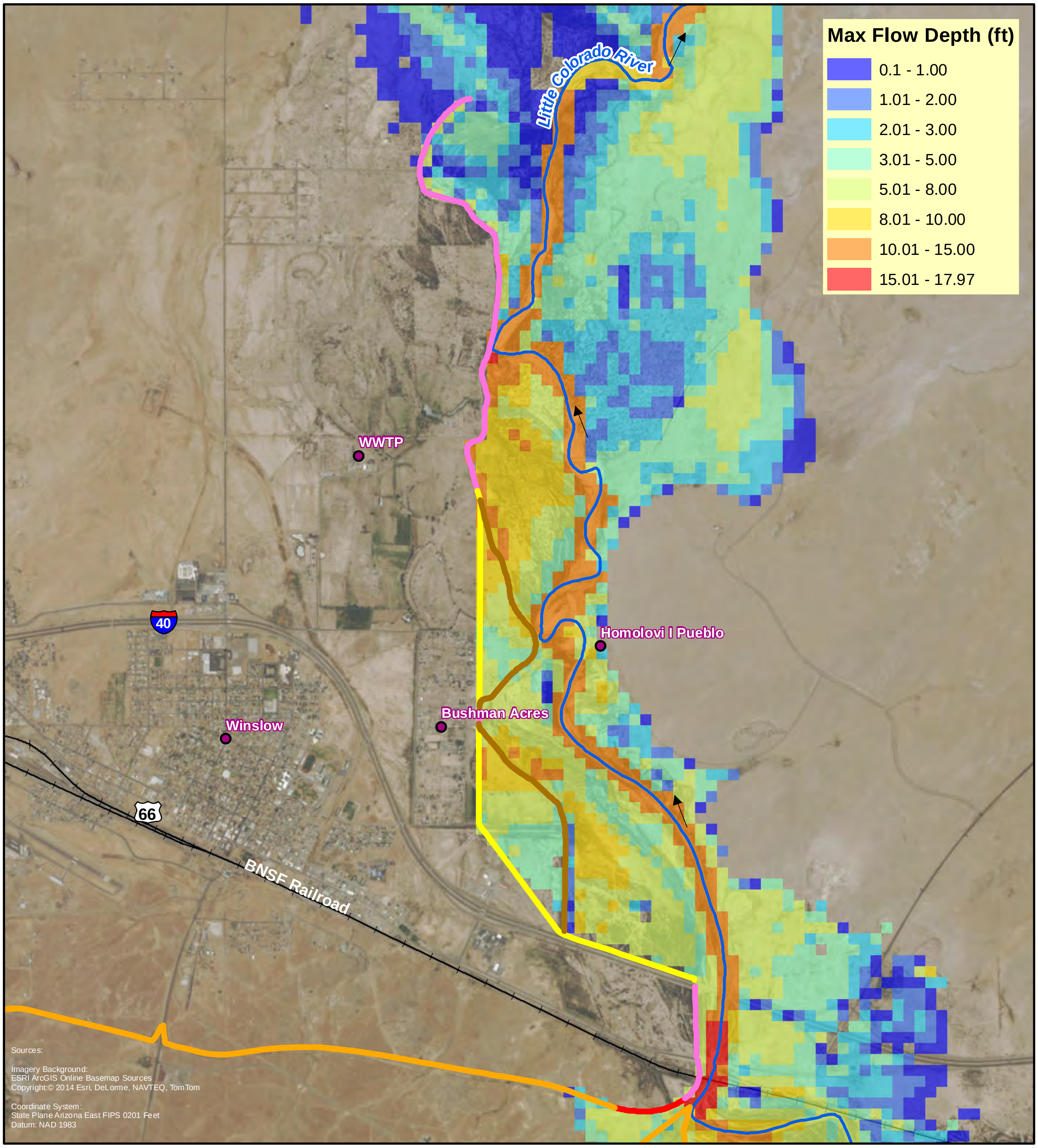
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 10
WITH-PROJECT CONDITION
1% ACE FLOOD
MAX FLOW DEPTH

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Rebuild Winslow Levee
- New Levee
- Remove Winslow Levee
- Rebuild RWDL
- RWDL - No Improvements
- Little Colorado River
- BNSF Railroad

0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

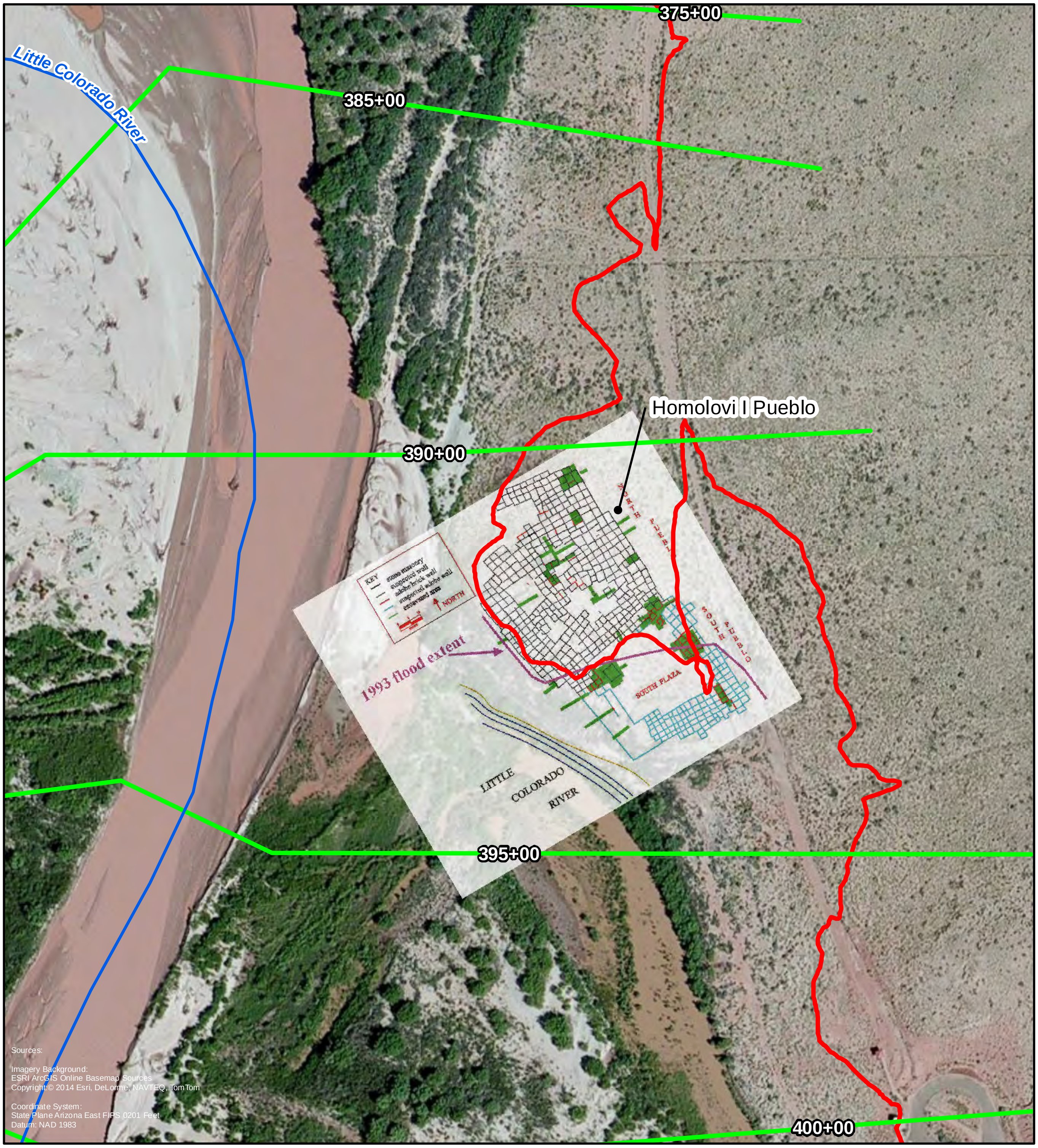
N

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 3.1
WITH-PROJECT CONDITION
1% ACE FLOOD
MAX FLOW DEPTH

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 42



Legend

- Little Colorado River
- Cross-Sections
- Baseline - 1% ACE Floodplain

N

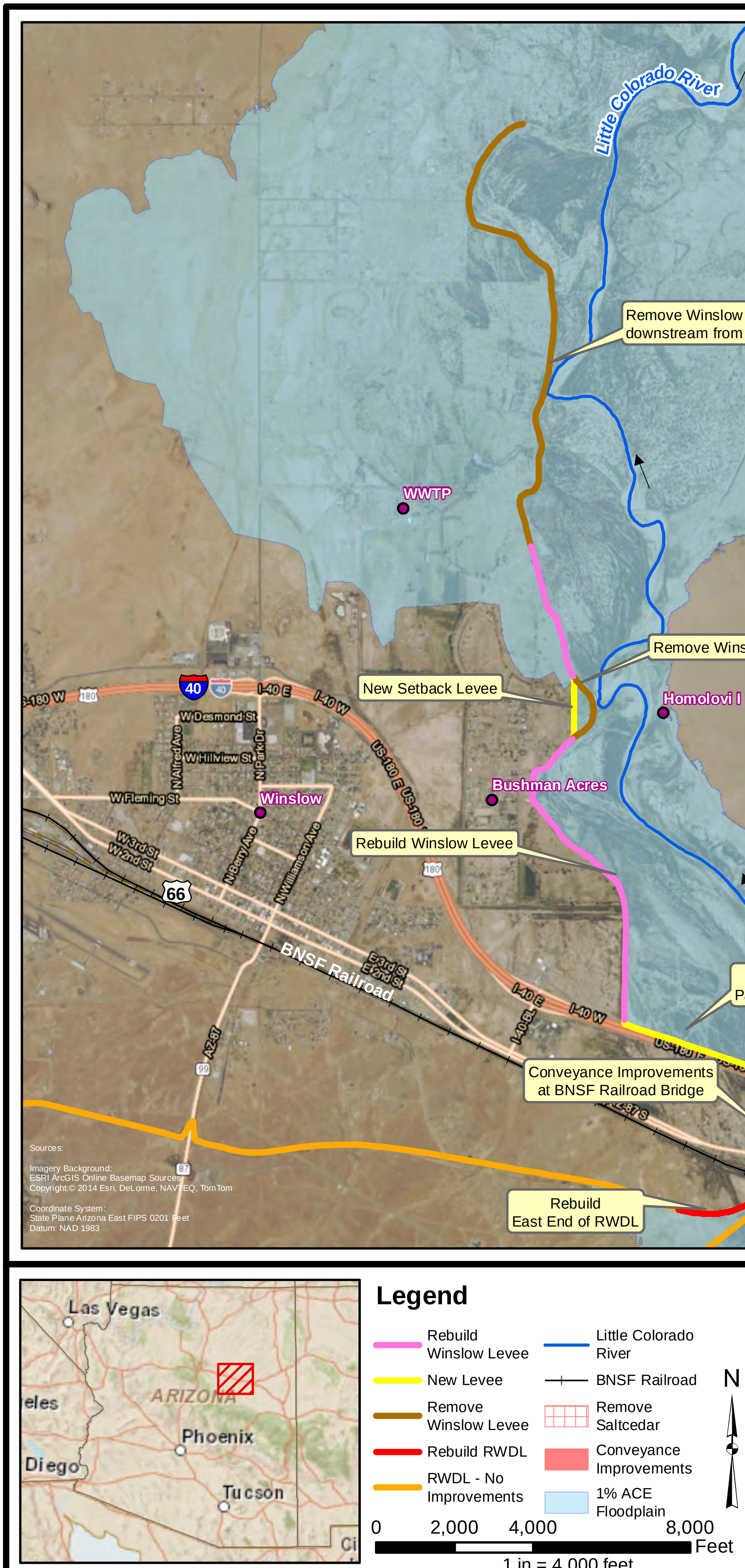
0 100 200 400 Feet

1 in = 200 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

COMPARISON OF BASELINE CONDITION 1% ACE FLOOD EVENT TO 1993 FLOOD EVENT HOMOLOVI I PUEBLO

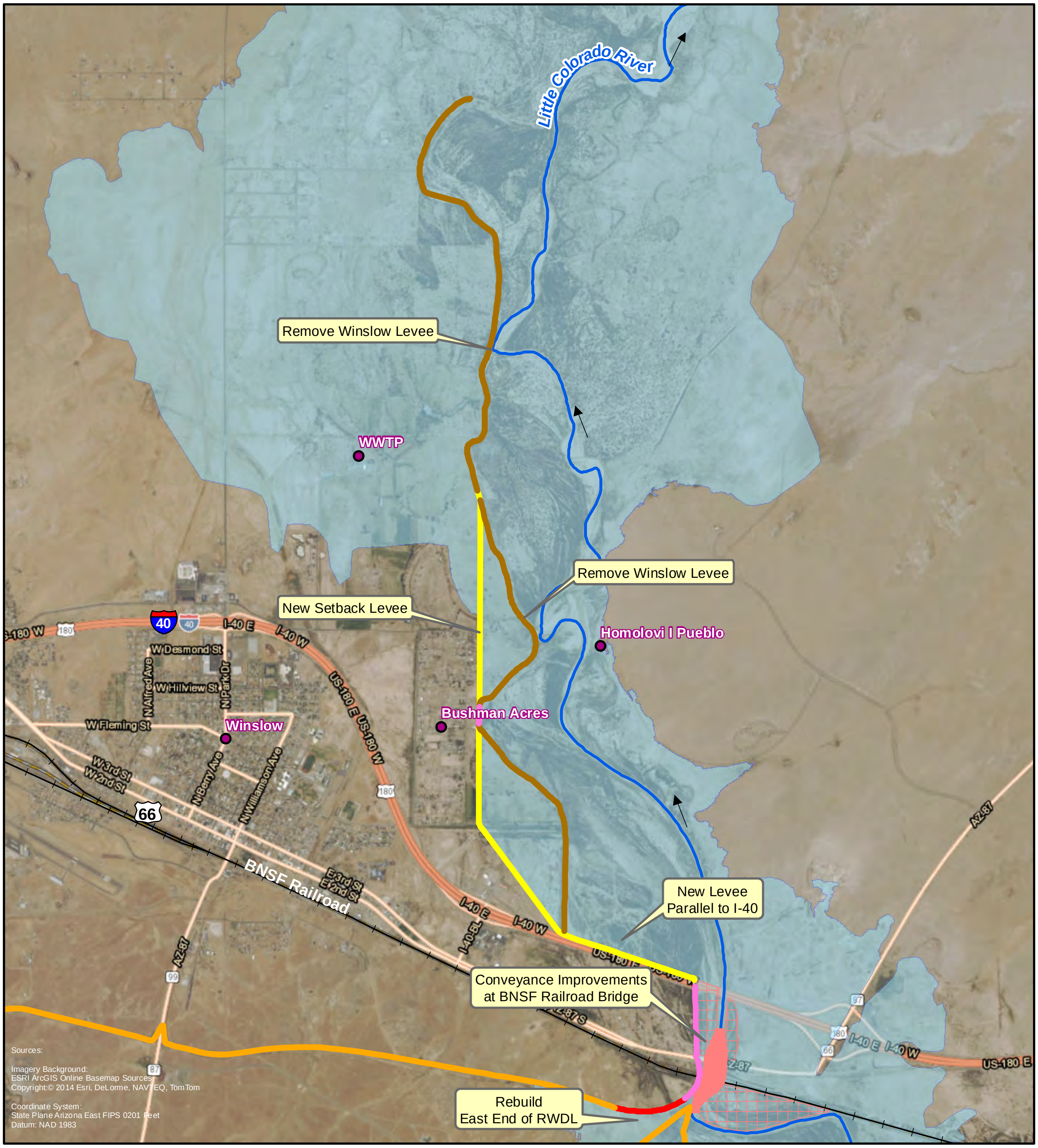
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

FLOOD RISK REDUCTION
MEASURE 1
ALTERNATIVE 10
WITH REMOVAL OF
WINSLOW LEVEE

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Remove Winslow Levee
- New Levee
- Rebuild Winslow Levee
- RWDL - No Improvements
- Rebuild RWDL
- Little Colorado River
- BNSF Railroad
- Conveyance Improvements
- Remove Saltcedar
- 1% ACE Floodplain

N

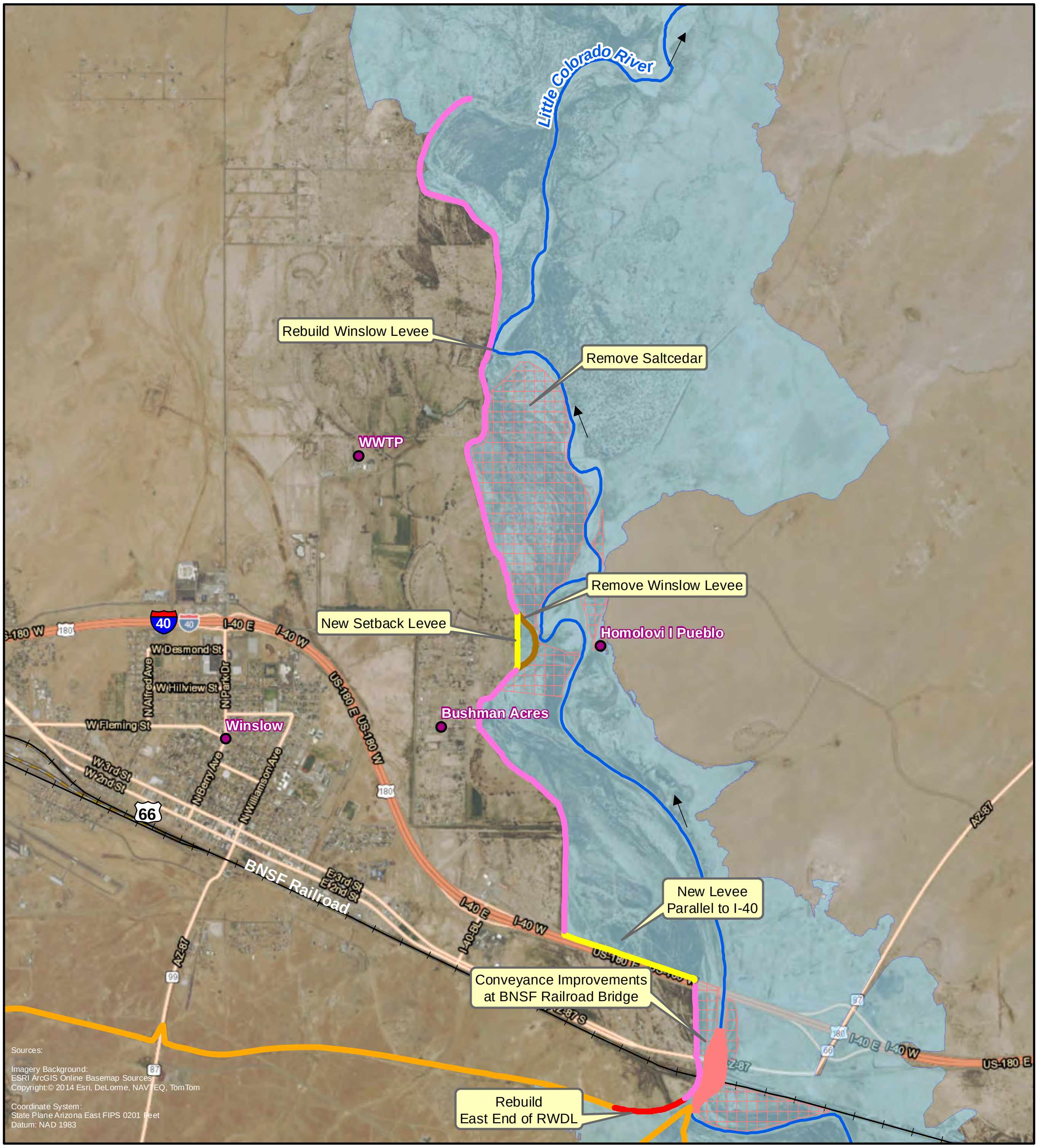
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

FLOOD RISK REDUCTION
MEASURE 2
ALTERNATIVE 3.1
WITHREMOVAL OF
WINSLOW LEVEE

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Remove Winslow Levee
- New Levee
- Rebuild Winslow Levee
- Rebuild RWDL
- RWDL - No Improvements
- Little Colorado River
- BNSF Railroad
- Remove Saltcedar
- Conveyance Improvements
- 1% ACE Floodplain

N

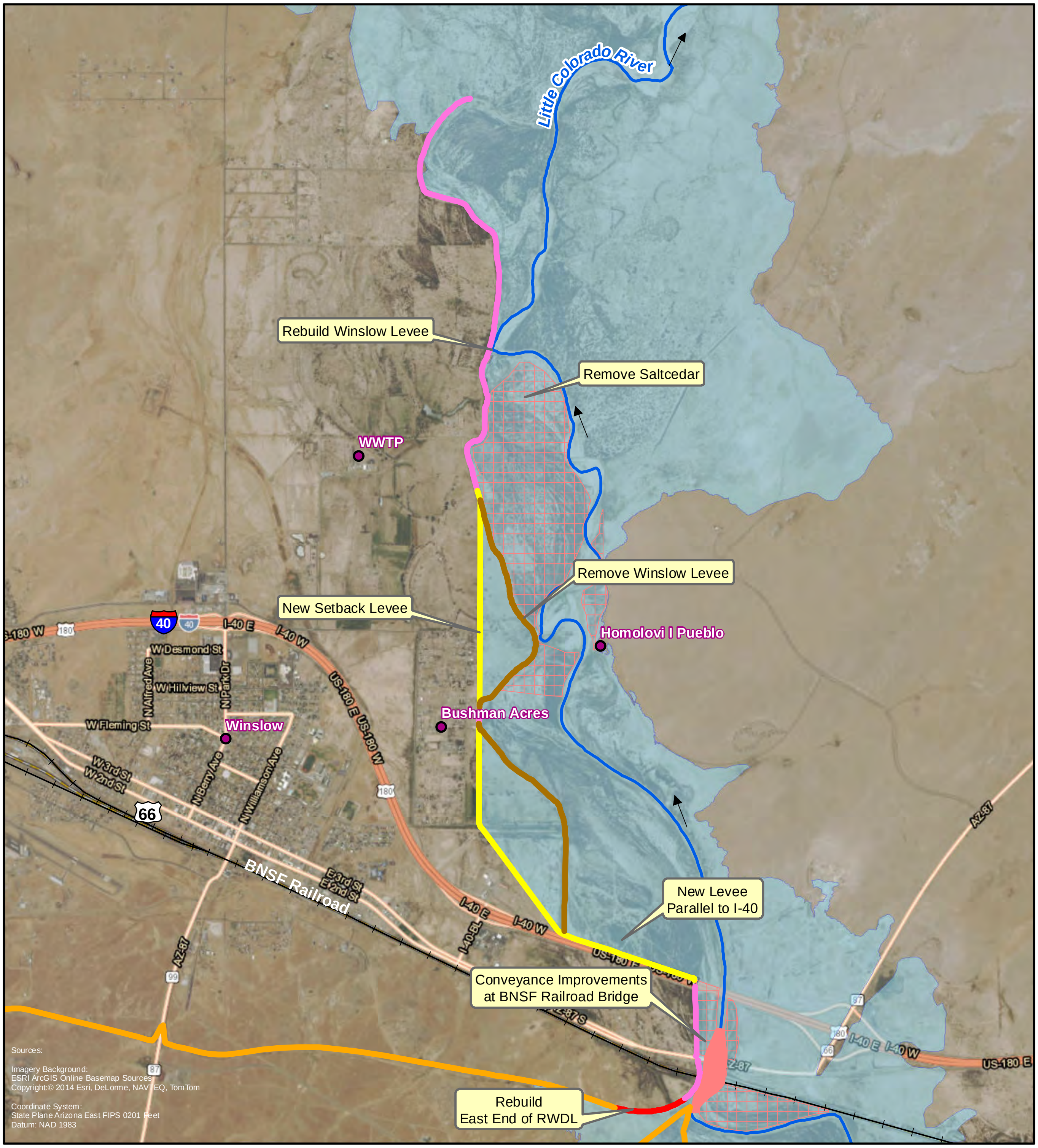
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

FLOOD RISK REDUCTION
MEASURE 3
ALTERNATIVE 8
WITH SALCEDAR
REMOVAL

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Remove Winslow Levee
- New Levee
- Rebuild Winslow Levee
- RWDL - No Improvements
- Rebuild RWDL
- Little Colorado River
- BNSF Railroad
- Conveyance Improvements
- Remove Saltcedar
- 1% ACE Floodplain

N

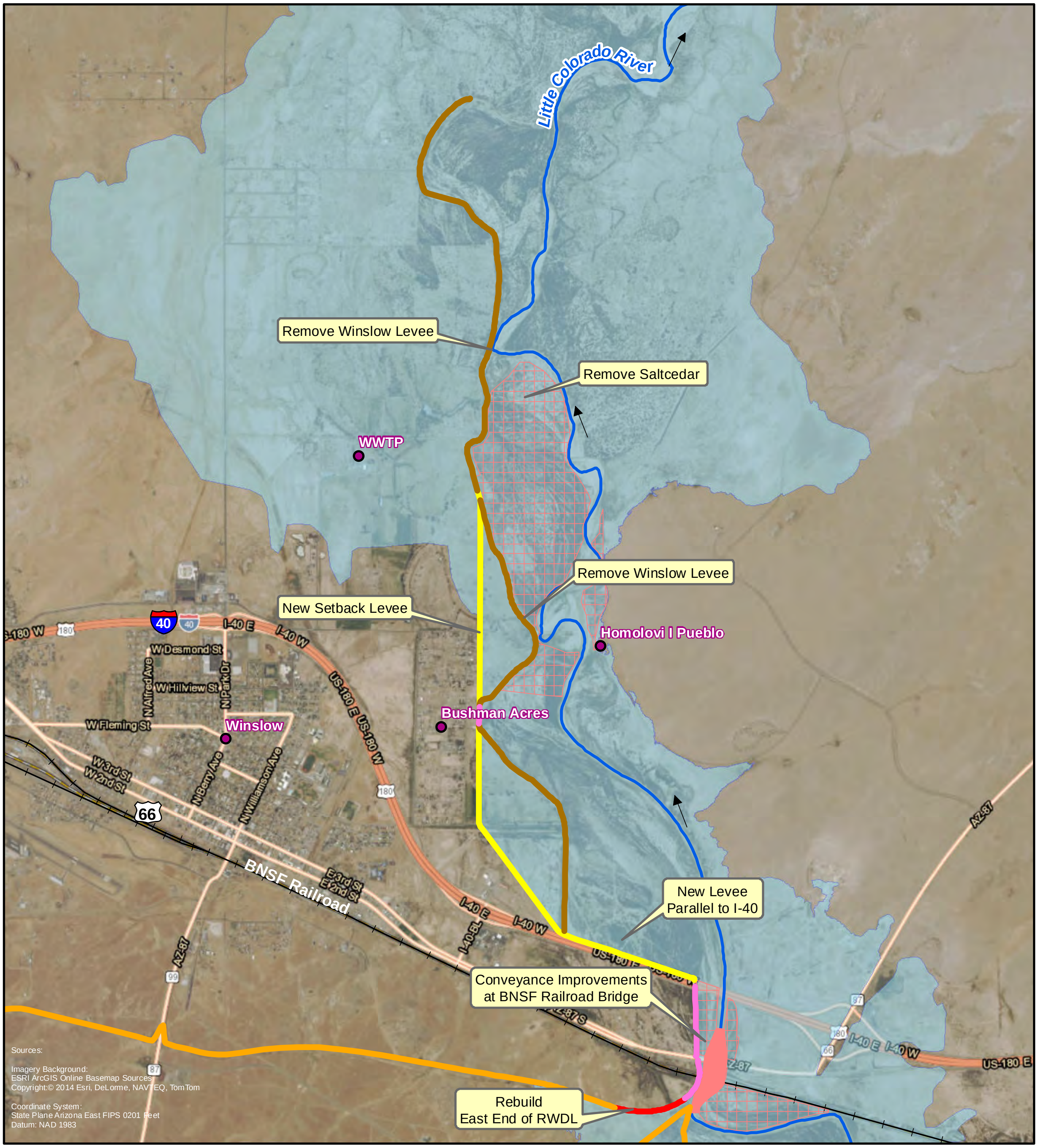
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

FLOOD RISK REDUCTION
MEASURE 4
ALTERNATIVE 3.1 WITH
SALTCEDAR REMOVAL
DOWNSTREAM FROM
HOMOLOVI I PUEBLO

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

Remove Winslow Levee	Little Colorado River
New Levee	BNSF Railroad
Rebuild Winslow Levee	Conveyance Improvements
RWDL - No Improvements	Remove Saltcedar
Rebuild RWDL	1% ACE Floodplain

N

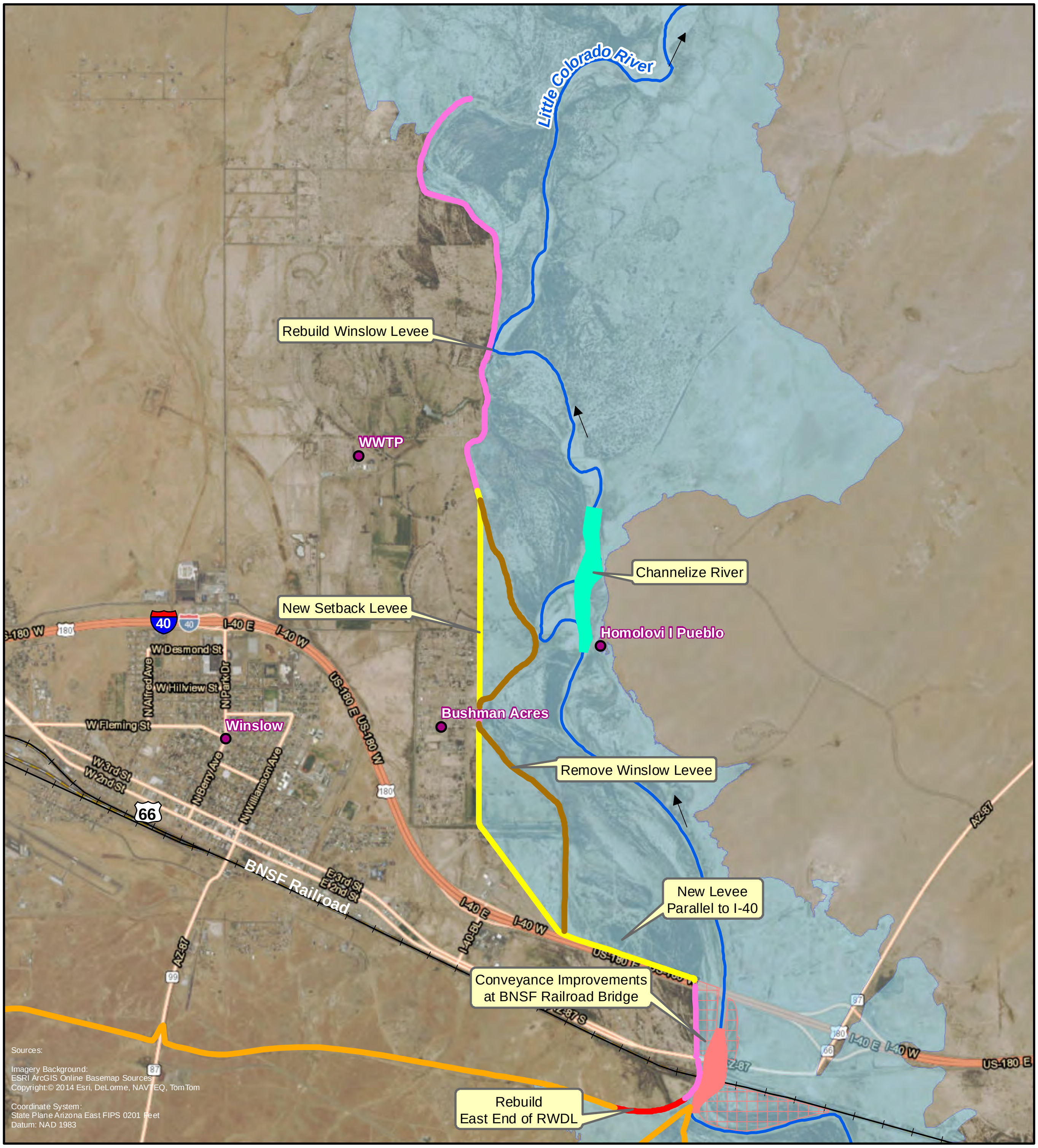
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

FLOOD RISK REDUCTION
MEASURE 5 ALTERNATIVE
3.1 WITH SALTCEDAR REMOVAL
DOWNSTREAM FROM
HOMOLOVI I PUEBLO &
REMOVAL OF WINSLOW LEVEE

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Remove Levee
- New Levee
- Rebuild Levee
- RWDL - No Improvements
- Rebuild RWDL
- Channelize LCR
- Little Colorado River
- BNSF Railroad
- Conveyance Improvements
- Remove Saltcedar
- 1% ACE Floodplain

N

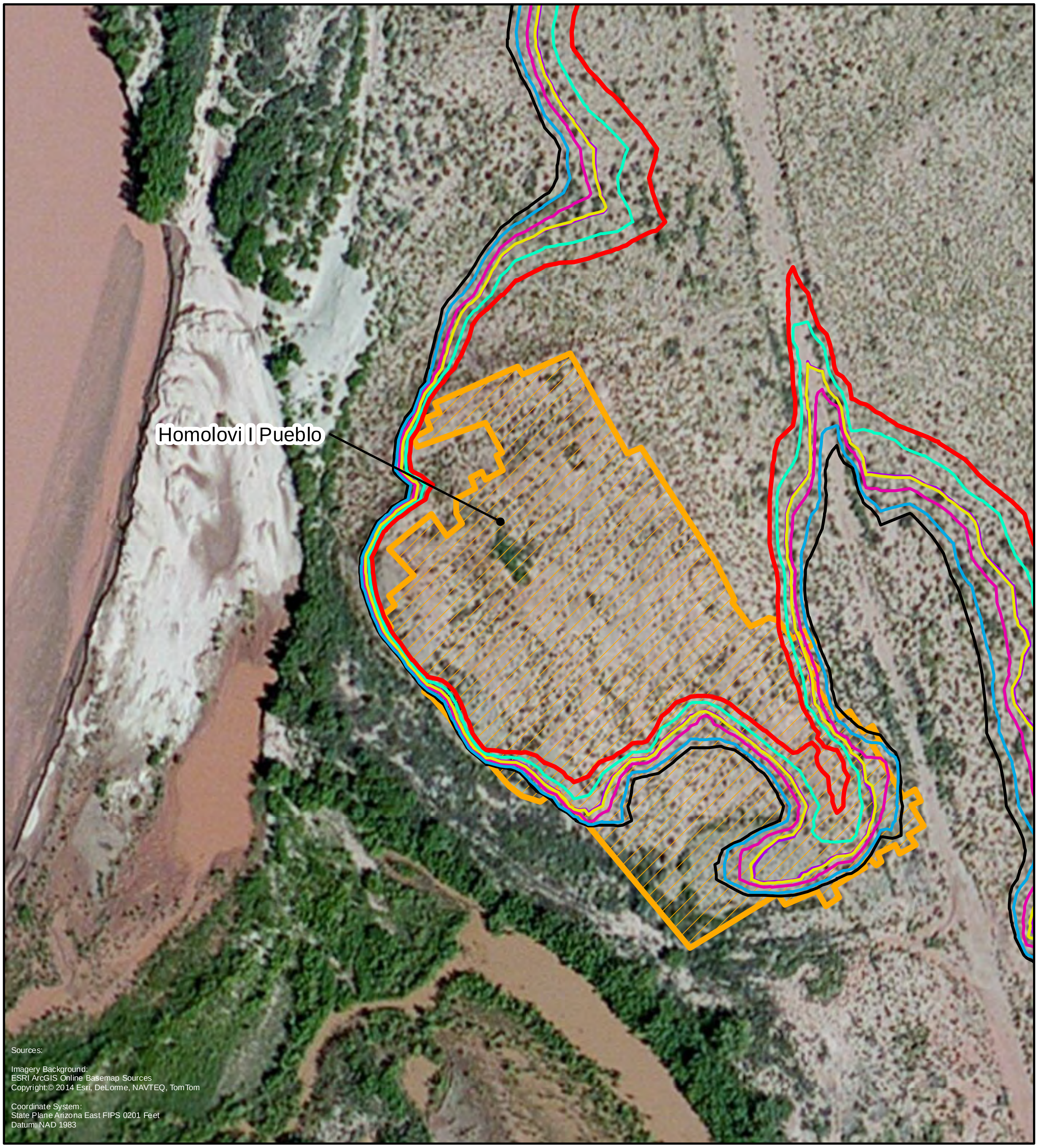
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

FLOOD RISK REDUCTION
MEASURE 6
ALTERNATIVE 3.1 WITH
CHANNELIZATION MEASURE
DOWNSTREAM FROM
HOMOLOVI I PUEBLO

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Baseline - 1% ACE Floodplain
- Homolovi I Pueblo Footprint
- FRRM 1 - 1% ACE Floodplain
- FRRM 2 - 1% ACE Floodplain
- FRRM 3 - 1% ACE Floodplain
- FRRM 4 - 1% ACE Floodplain
- FRRM 5 - 1% ACE Floodplain
- FRRM 6 - 1% ACE Floodplain

N

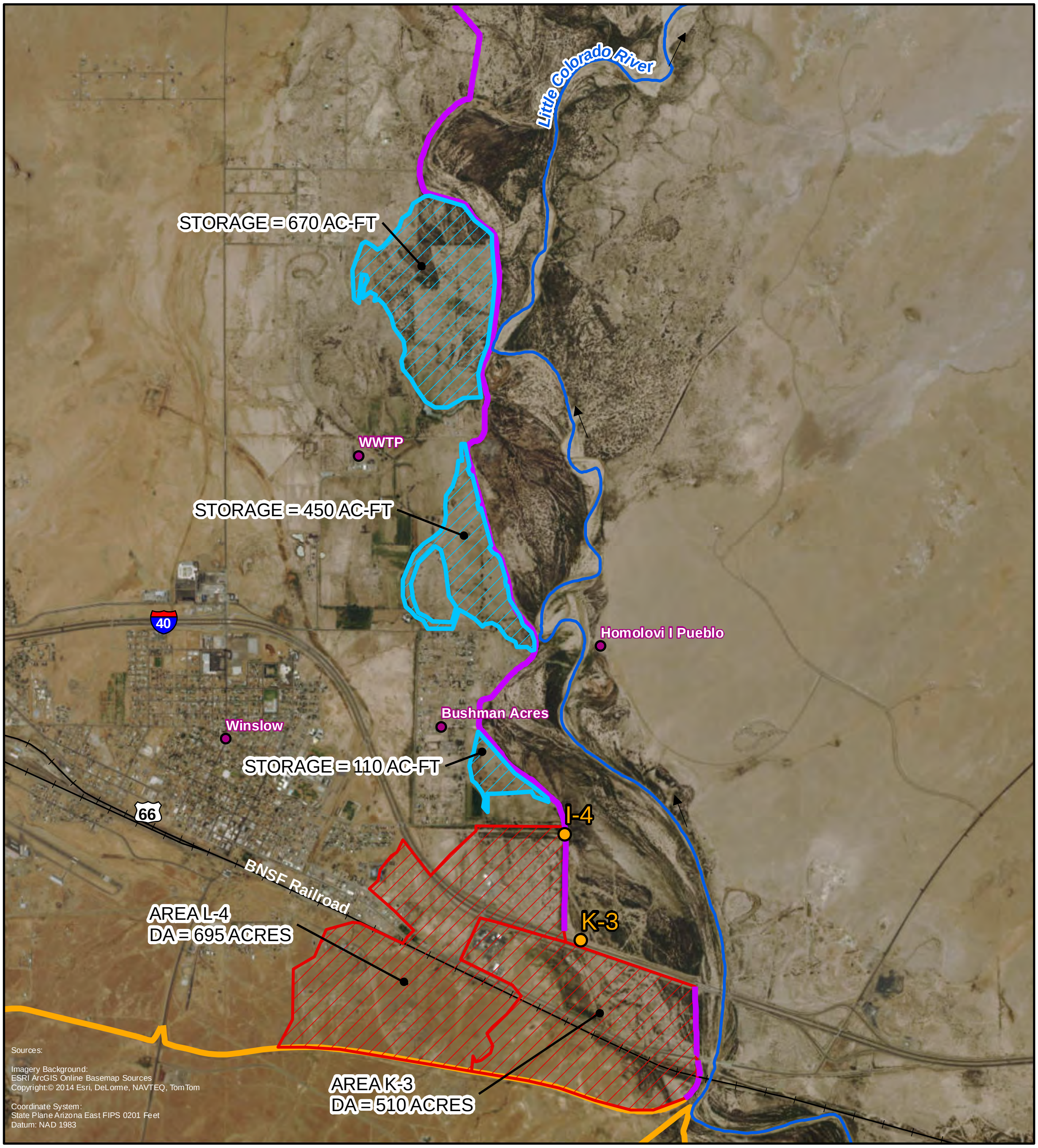
0 50 100 200 Feet

1 in = 100 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

COMPARISON OF FLOOD
RISK REDUCTION MEASURES
TO BASELINE
AT HOMOLOVI I PUEBLO
1% ACE FLOOD

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad
- Hydraulic Gates
- Ponding Area
- Drainage Area

N

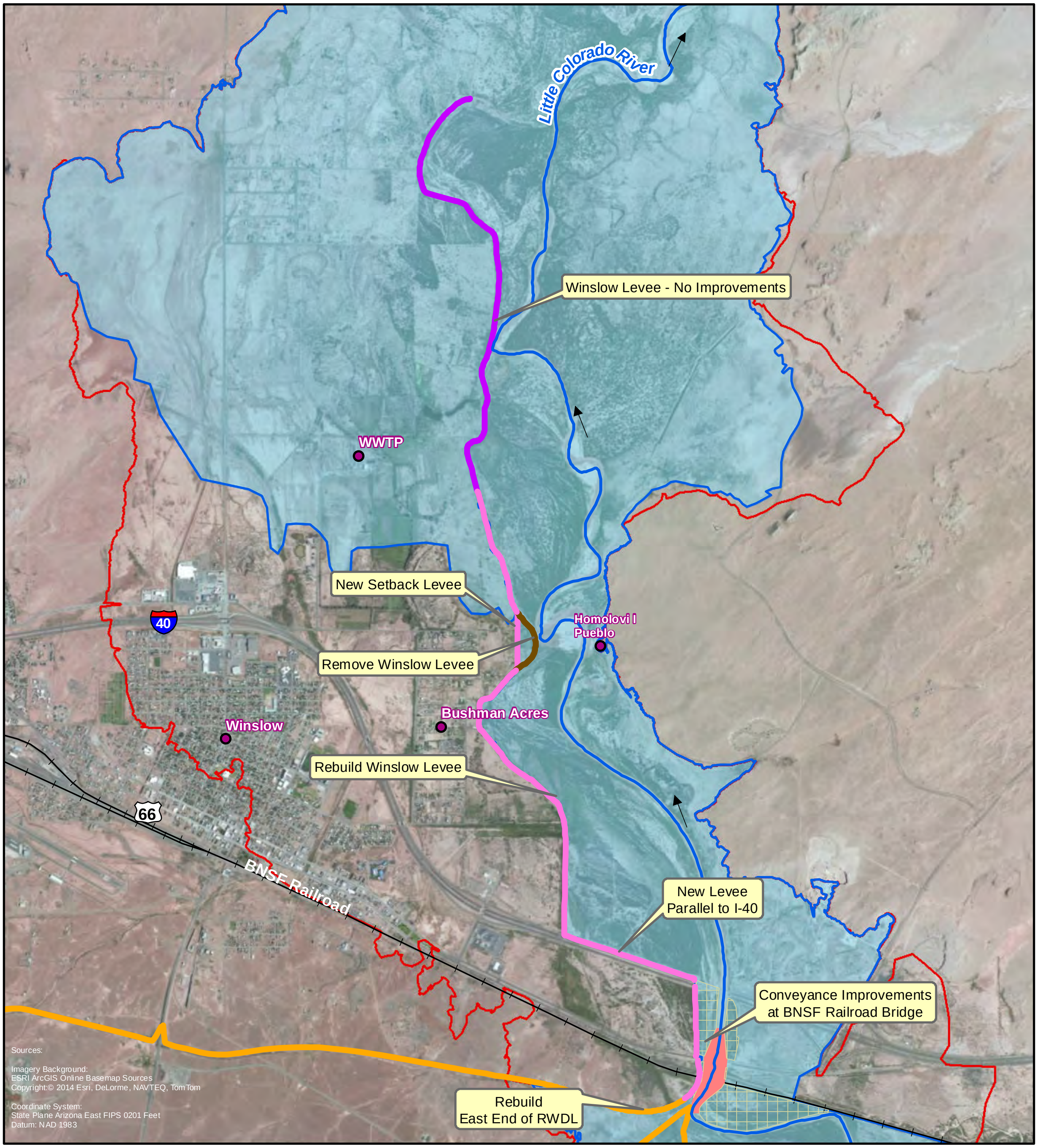
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

WINSLOW LEVEE
INTERIOR DRAINAGE MAP

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Winslow Levee
- Remove Winslow Levee
- Winslow Levee - No Improvements
- RWDL
- Remove Saltcedar
- Conveyance Improvements
- BNSF Railroad
- TSP 1% ACE Floodplain
- Baseline Condition 1% ACE Floodplain

N

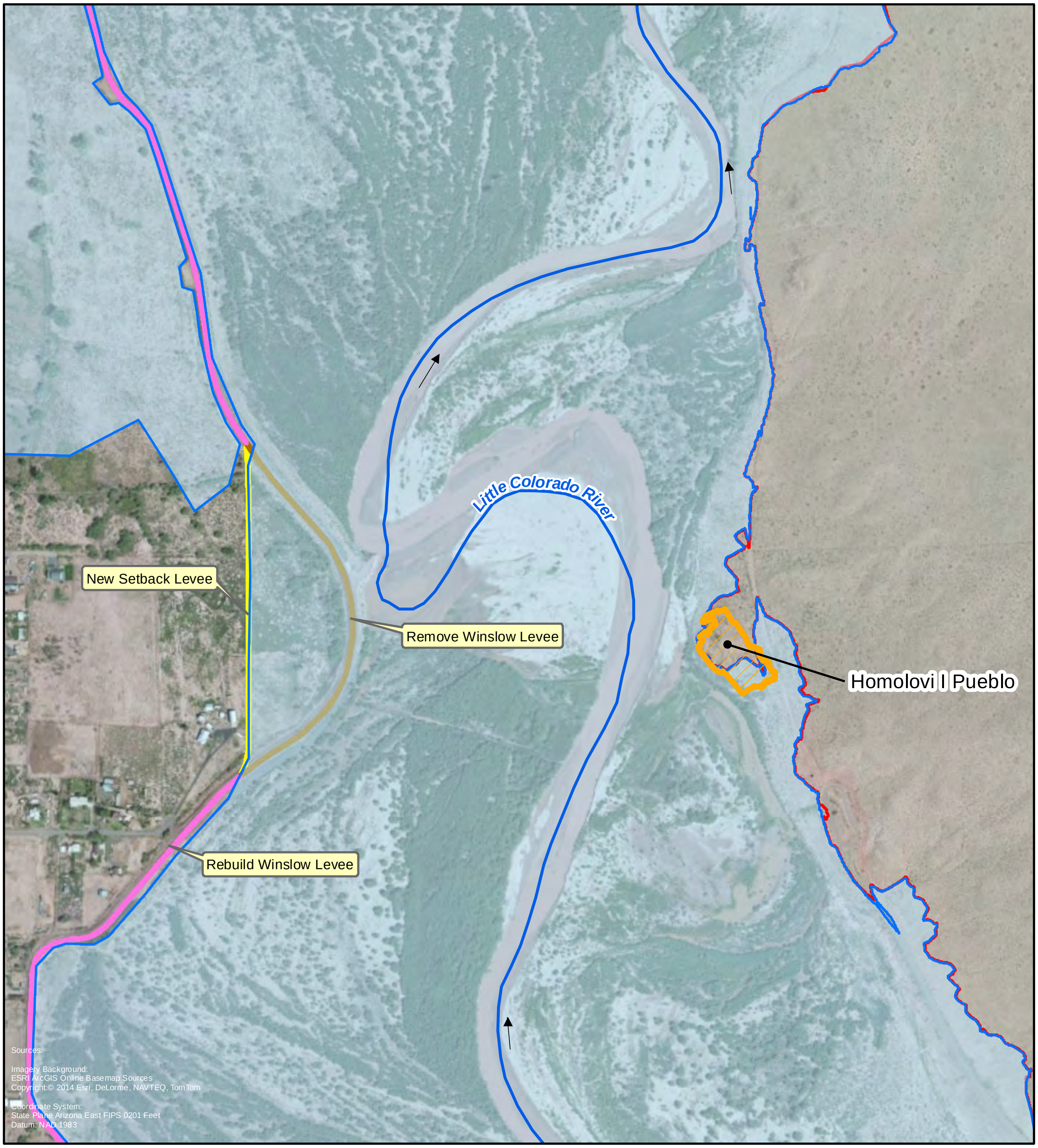
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

COMPARISON OF ALTERNATIVE 10.1
(TENTATIVELY SELECTED PLAN) &
BASELINE CONDITION
1% ACE FLOODPLAINS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Homolovi I Pueblo Footprint
- TSP - 1% ACE Floodplain
- Baseline - 1% ACE Floodplain
- Rebuild Winslow Levee
- Remove Winslow Levee
- New Levee

N

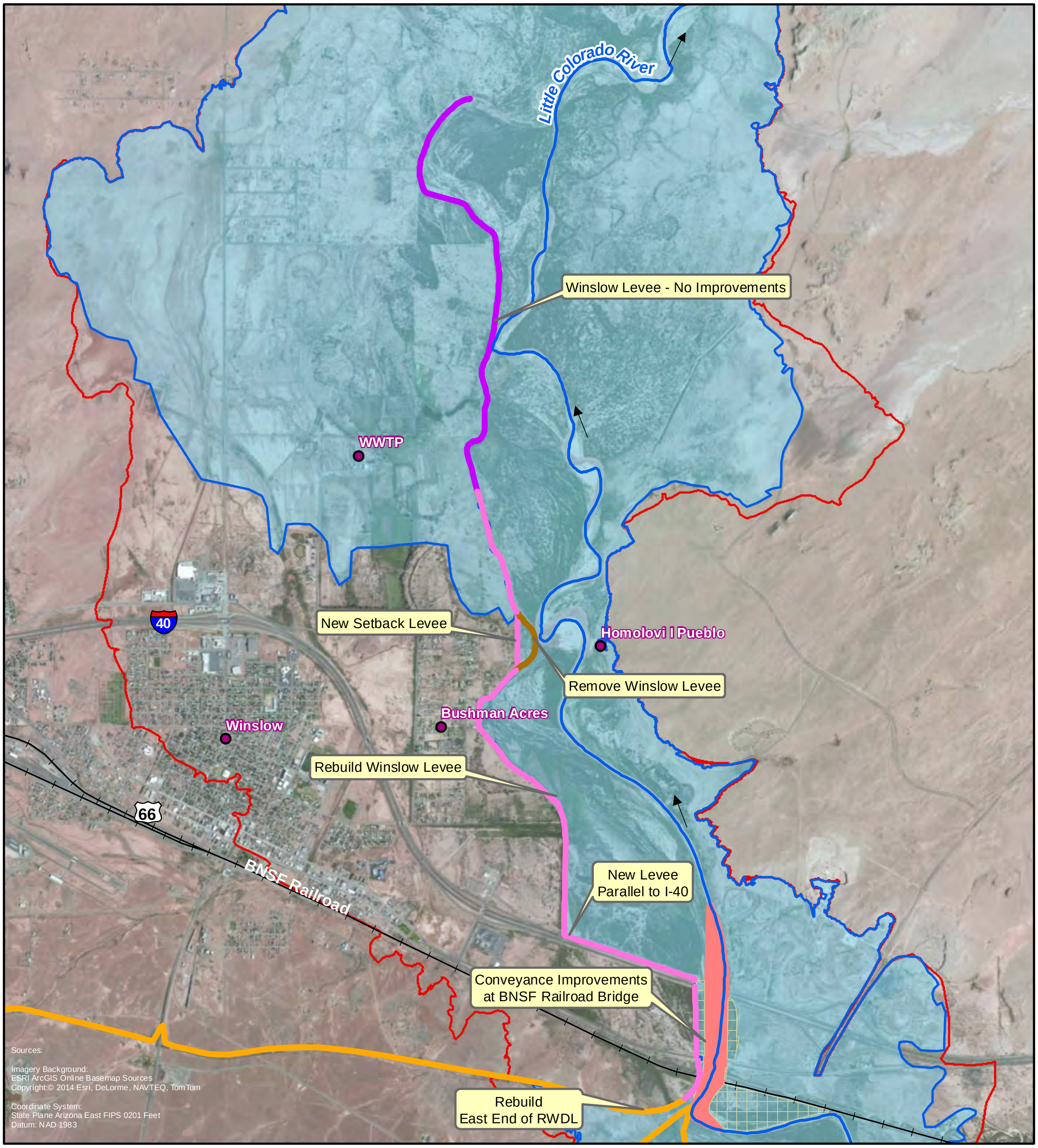
0 350 700 1,400 Feet

1 in = 700 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

TENTATIVELY SELECTED PLAN
FLOODPLAIN NEAR
HOMOLOVI I PUEBLO
1 % ACE FLOOD

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Winslow Levee
- Remove Winslow Levee
- Winslow Levee - No Improvements
- RWDL
- Remove Saltcedar
- Conveyance Improvements
- BNSF Railroad
- Alternative 10.4 - 0.5% ACE Floodplain
- Baseline Condition 0.5% ACE Floodplain

N

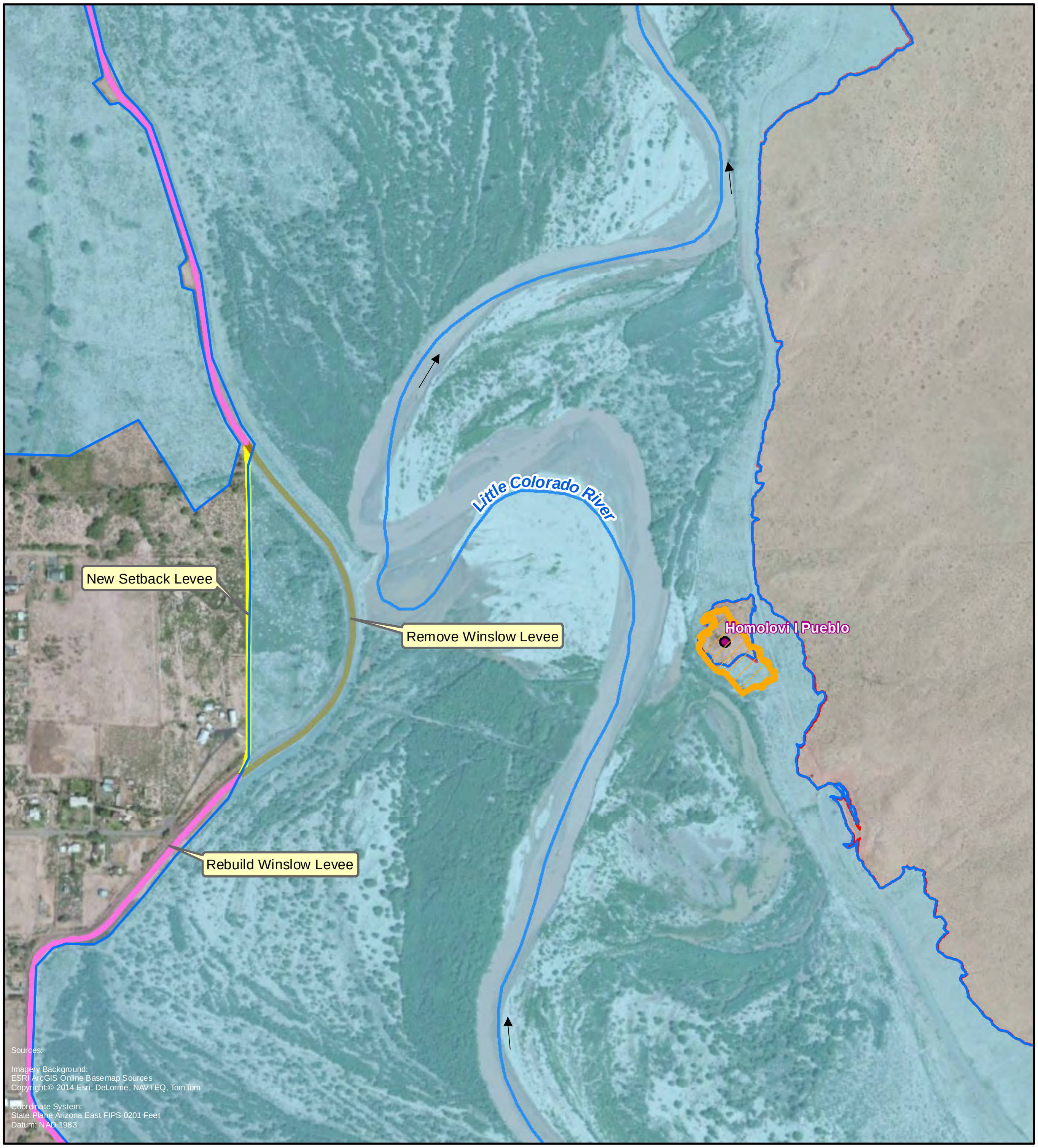
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

COMPARISON OF ALTERNATIVE 10.4
(0.5% ACE FLOODPLAIN) AND
BASELINE CONDITION
(0.5% ACE FLOODPLAIN)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Homolovi I Pueblo Footprint
- Alternative 10.4 - 0.5% ACE Floodplain
- Alternative 10.4 - 0.5% ACE Floodplain
- Baseline - 0.5% ACE Floodplain
- Little Colorado River
- Rebuild Winslow Levee
- Remove Winslow Levee
- New Levee

N

0 350 700 1,400 Feet

1 in = 700 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

COMPARISON OF
ALTERNATIVE 10.4
(0.5% ACE FLOODPLAIN) TO
BASELINE CONDITION
(0.5% ACE FLOODPLAIN)
NEAR HOMOLOVI I PUEBLO

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

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FIGURES

LITTLE COLORADO RIVER AT WINSLOW
HYDRAULIC AND SEDIMENTATION APPENDIX
APRIL 2016

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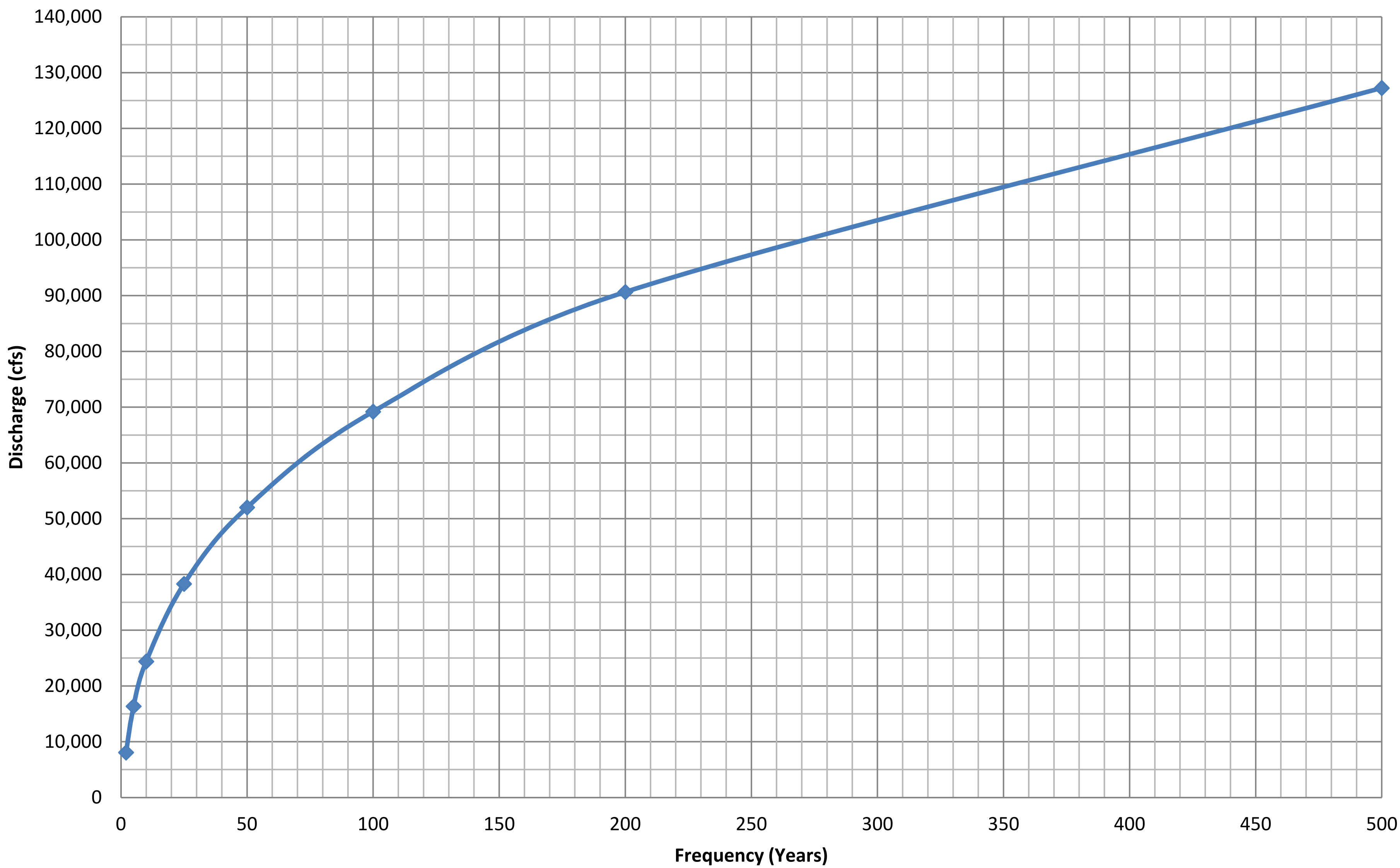
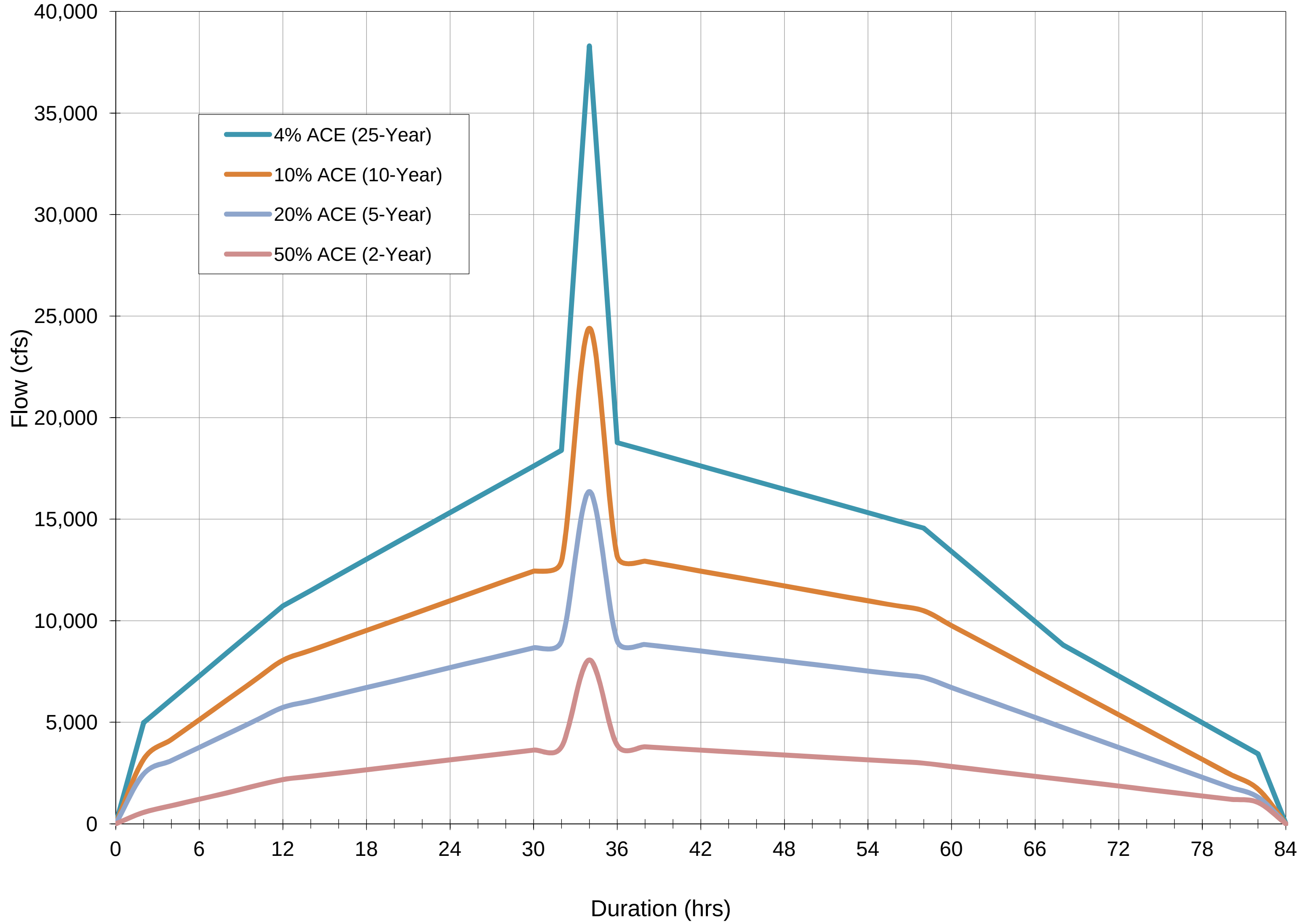


HYDRAULIC AND SEDIMENTATION APPENDIX
LITTLE COLORADO RIVER
WINSLOW, ARIZONA

LITTLE COLORADO RIVER
DISCHARGE – FREQUENCY CURVE
LCR near Winslow (USGS gage 09400350)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

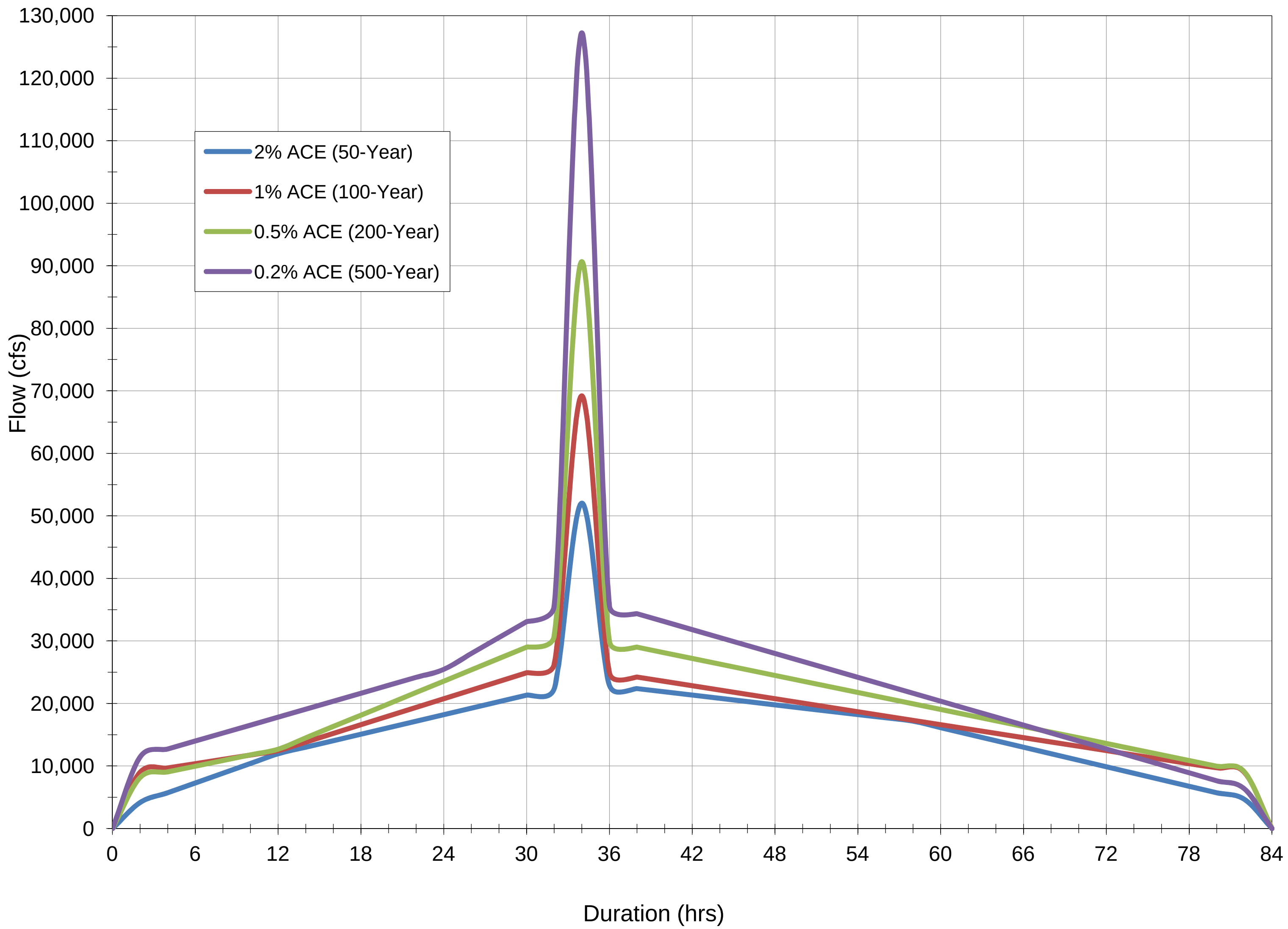
FIGURE 1



Notes: Drainage Area = 16,192 square miles; Hydrograph Data from USGS - 09400350

HYDRAULIC AND SEDIMENTATION APPENDIX LITTLE COLORADO RIVER WINSLOW, ARIZONA	**LITTLE COLORADO RIVER 50%, 20%, 10%, and 4% ACE FLOODS BALANCED HYDROGRAPHS**	U.S. ARMY CORPS OF ENGINEERS LOS ANGELES DISTRICT

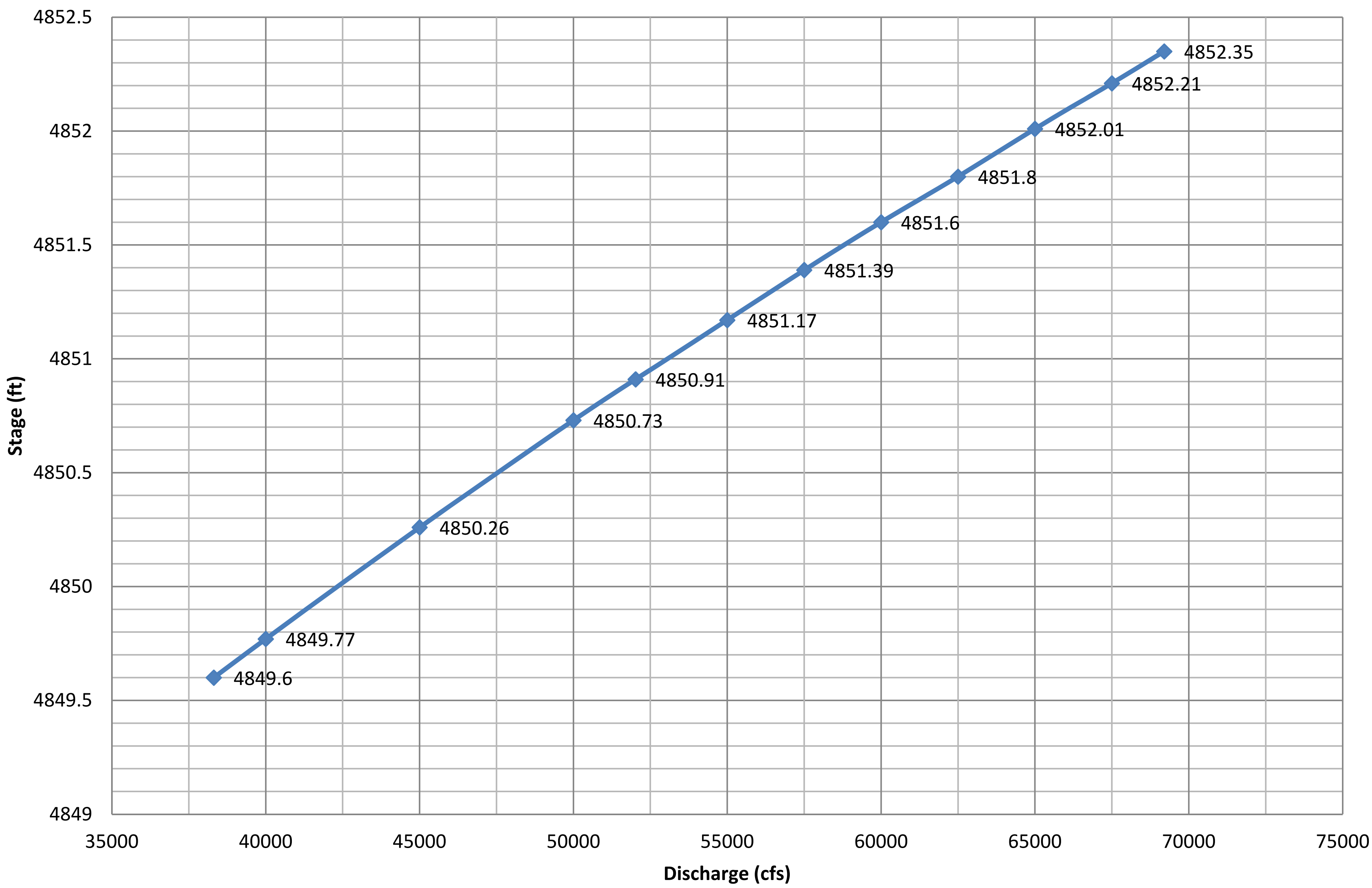
FIGURE 2



Notes: Drainage Area = 16,192 square miles; Hydrograph Data from USGS - 09400350

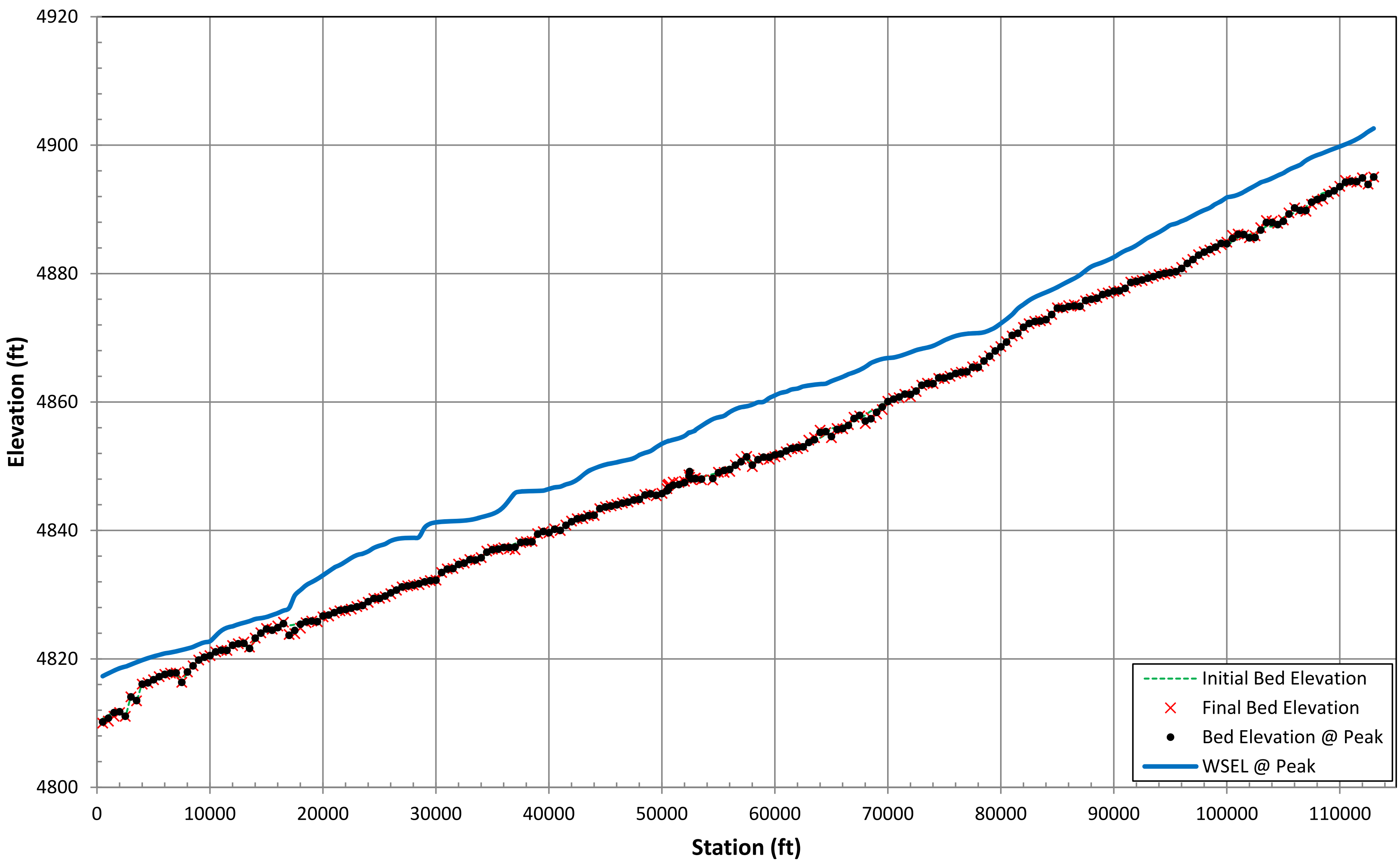
HYDRAULIC AND SEDIMENTATION APPENDIX LITTLE COLORADO RIVER WINSLOW, ARIZONA	**LITTLE COLORADO RIVER 2%, 1%, 0.5%, and 0.2% ACE FLOODS BALANCED HYDROGRAPHS**	U.S. ARMY CORPS OF ENGINEERS LOS ANGELES DISTRICT

FIGURE 3



HYDRAULIC AND SEDIMENTATION APPENDIX LITTLE COLORADO RIVER WINSLOW, ARIZONA	**LITTLE COLORADO RIVER EXISTING CONDITION STAGE DISCHARGE CURVE AT HOMOLOVI I PUEBLO (STA 390+00)**	U.S. ARMY CORPS OF ENGINEERS LOS ANGELES DISTRICT

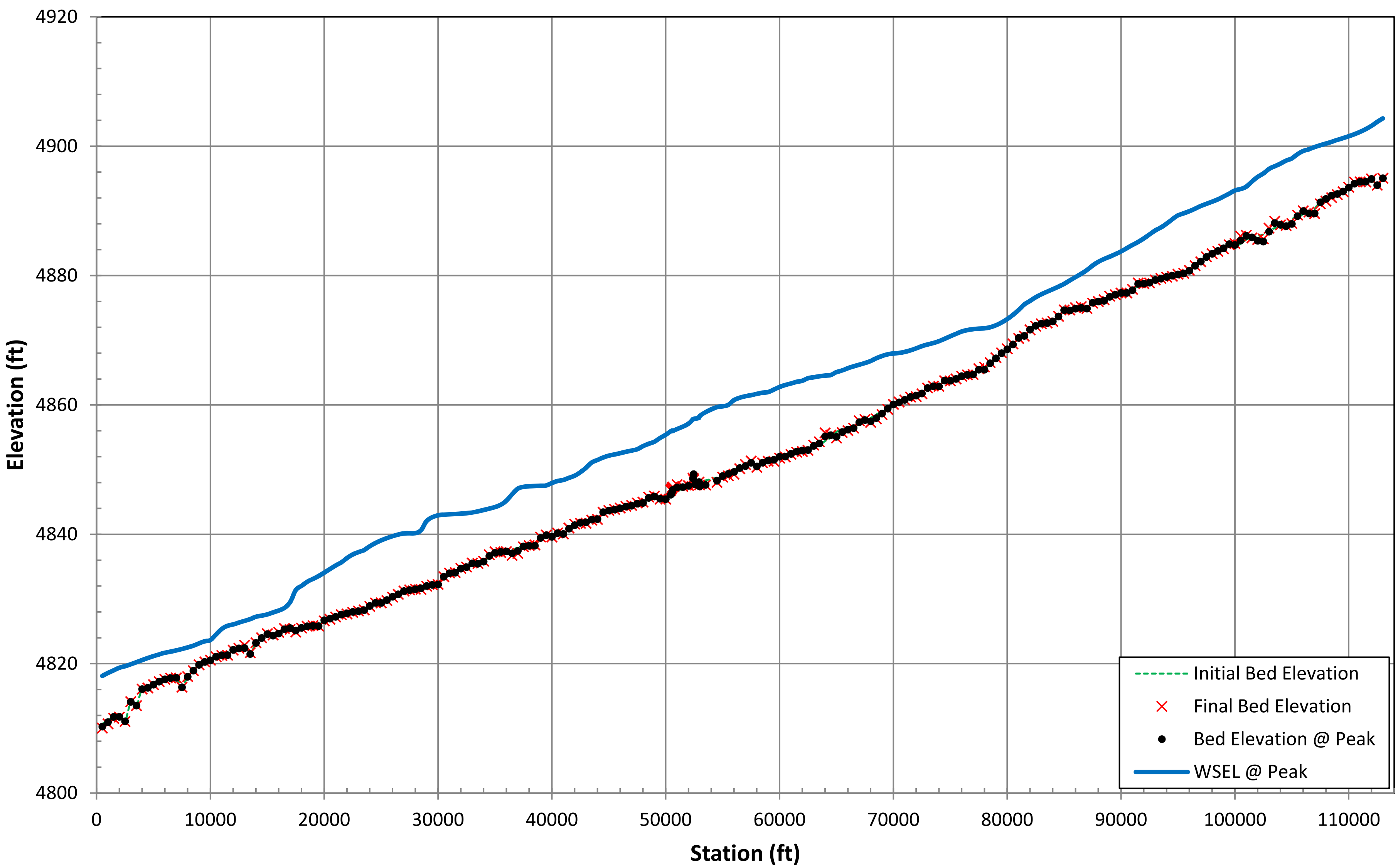
FIGURE 4



Note: LCR sediment transport results from HEC-RAS using n-year hydrographs.

HYDRAULIC AND SEDIMENTATION APPENDIX LITTLE COLORADO RIVER WINSLOW, ARIZONA	**LITTLE COLORADO RIVER BED PROFILES AFTER 50% ACE FLOOD**	U.S. ARMY CORPS OF ENGINEERS LOS ANGELES DISTRICT

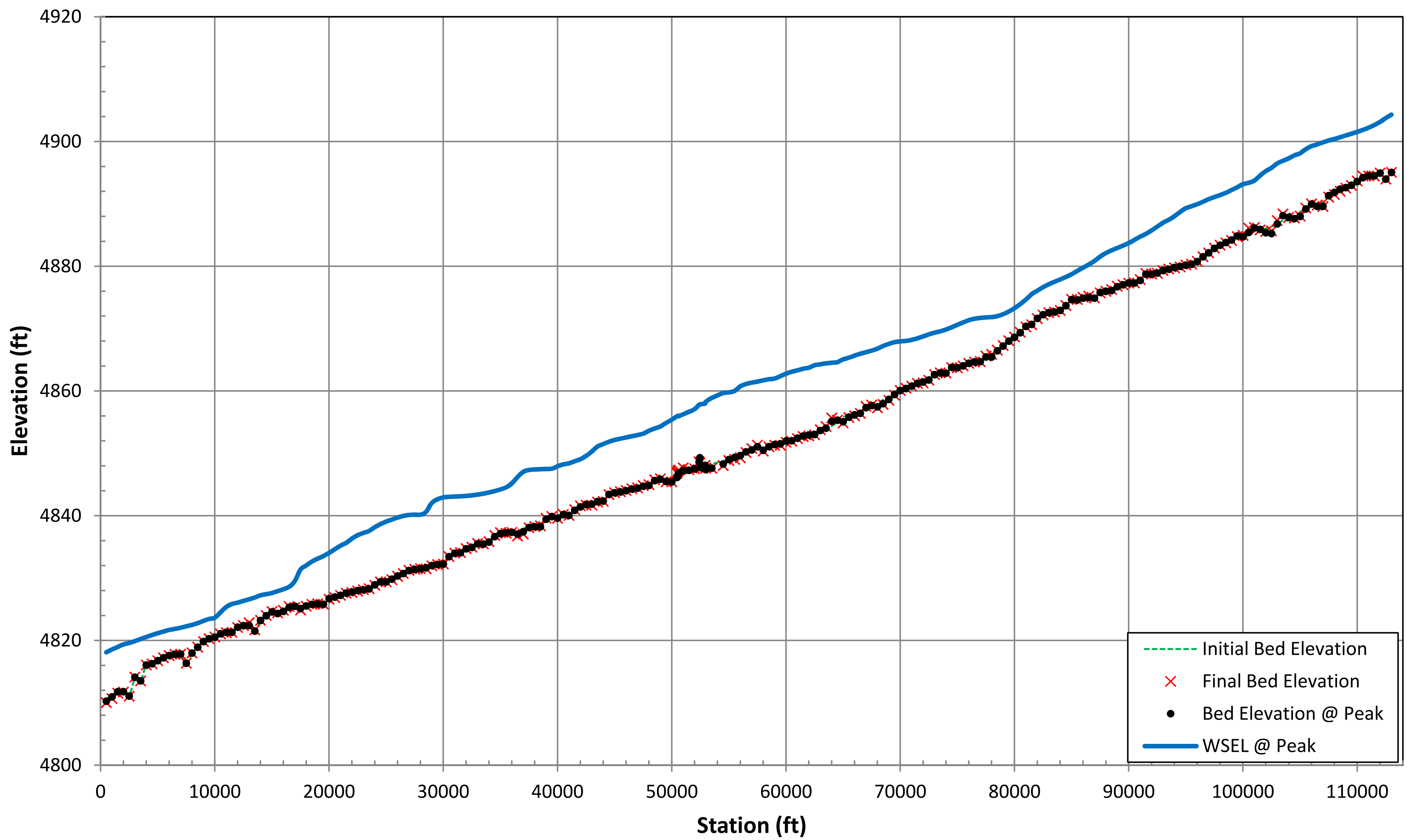
FIGURE 5



Note: LCR sediment transport results from HEC-RAS using n-year hydrographs.

HYDRAULIC AND SEDIMENTATION APPENDIX LITTLE COLORADO RIVER WINSLOW, ARIZONA	**LITTLE COLORADO RIVER BED PROFILES AFTER 20% ACE FLOOD**	U.S. ARMY CORPS OF ENGINEERS LOS ANGELES DISTRICT

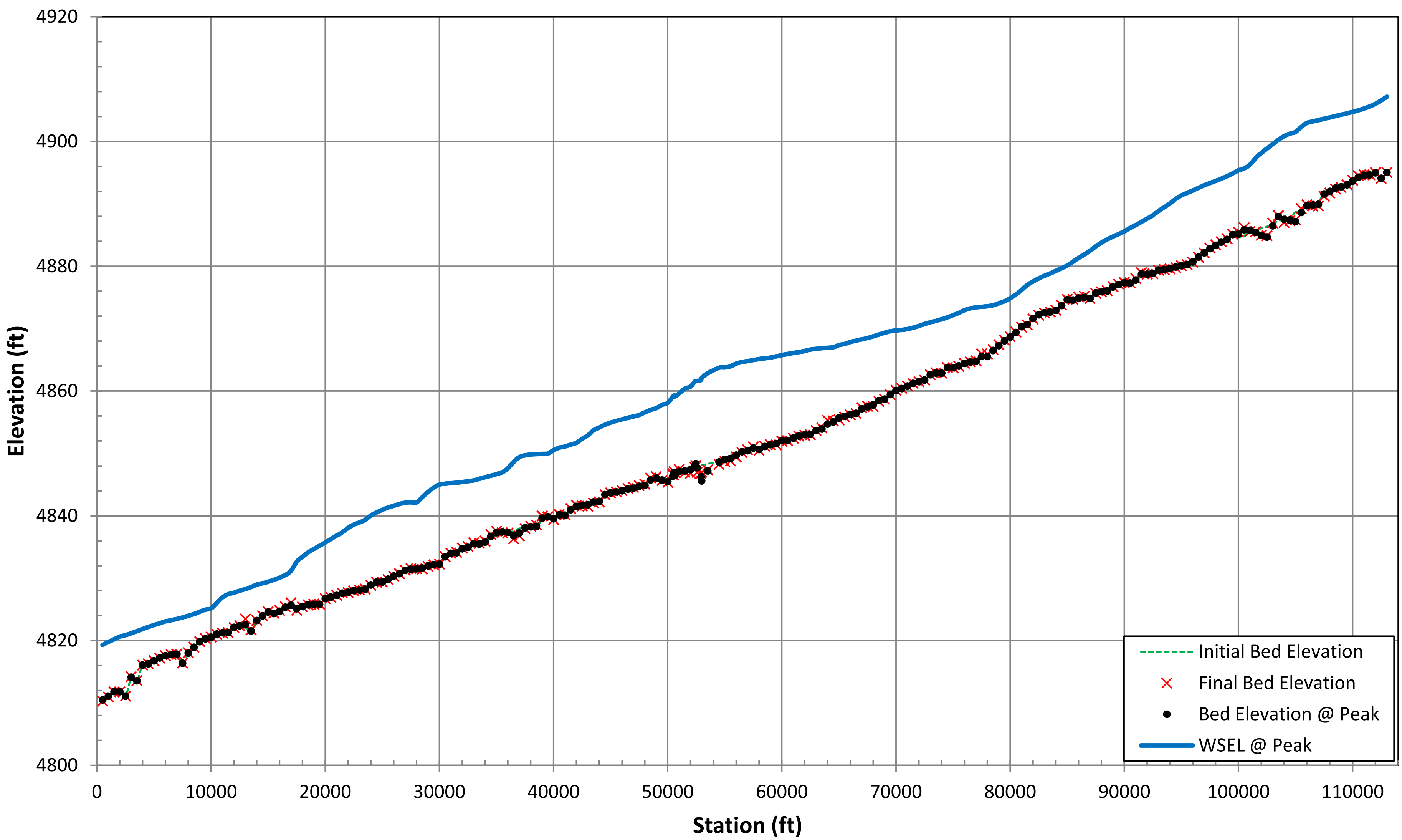
FIGURE 6



Note: LCR sediment transport results from HEC-RAS using n-year hydrographs.

HYDRAULIC AND SEDIMENTATION APPENDIX LITTLE COLORADO RIVER WINSLOW, ARIZONA	**LITTLE COLORADO RIVER** **BED PROFILES** **AFTER 10% ACE FLOOD**		U.S. ARMY CORPS OF ENGINEERS LOS ANGELES DISTRICT

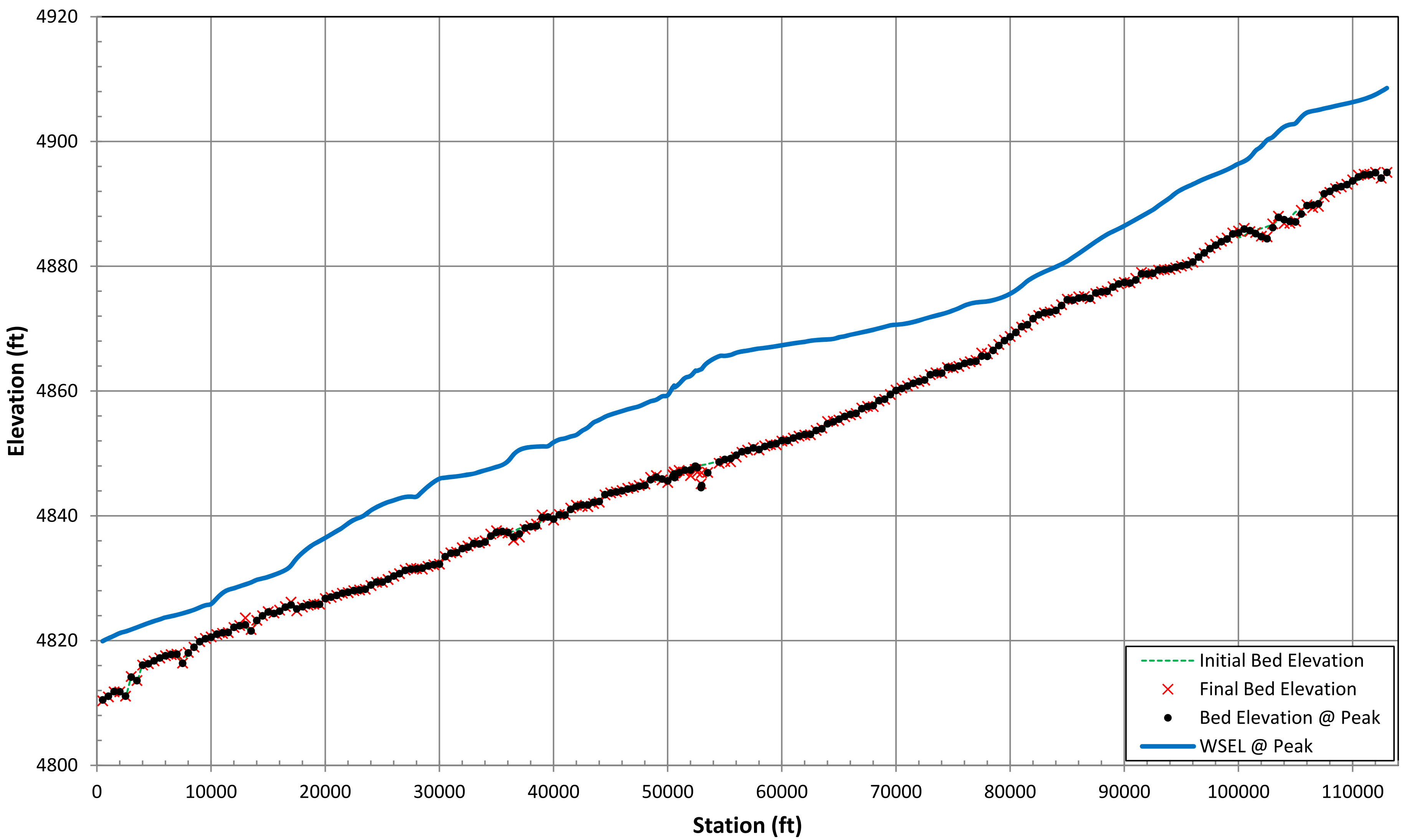
FIGURE 7



Note: LCR sediment transport results from HEC-RAS using n-year hydrographs.

HYDRAULIC AND SEDIMENTATION APPENDIX LITTLE COLORADO RIVER WINSLOW, ARIZONA	**LITTLE COLORADO RIVER BED PROFILES AFTER 4% ACE FLOOD**	U.S. ARMY CORPS OF ENGINEERS LOS ANGELES DISTRICT

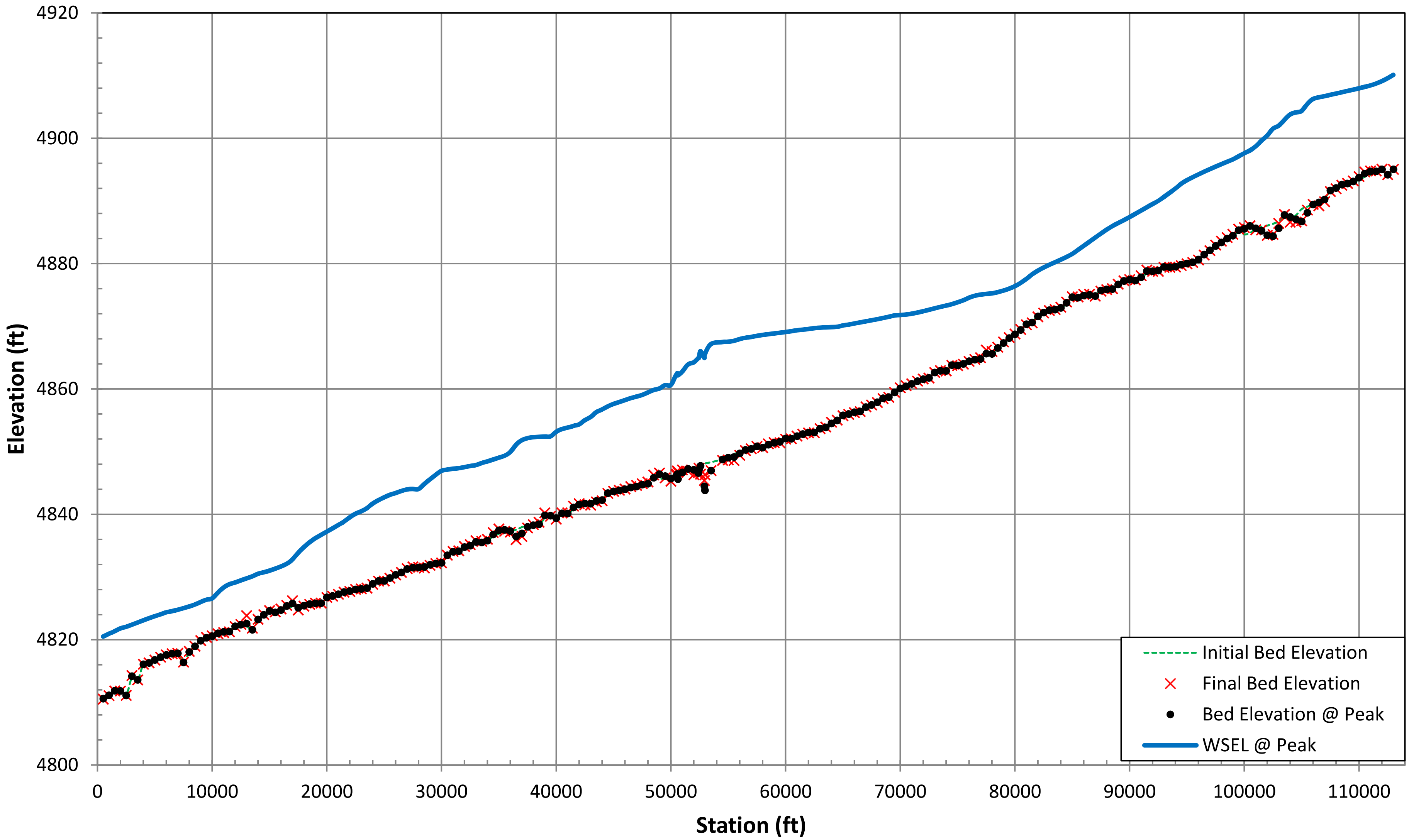
FIGURE 8



Note: LCR sediment transport results from HEC-RAS using n-year hydrographs.

HYDRAULIC AND SEDIMENTATION APPENDIX LITTLE COLORADO RIVER WINSLOW, ARIZONA	**LITTLE COLORADO RIVER BED PROFILES AFTER 2% ACE FLOOD**	U.S. ARMY CORPS OF ENGINEERS LOS ANGELES DISTRICT

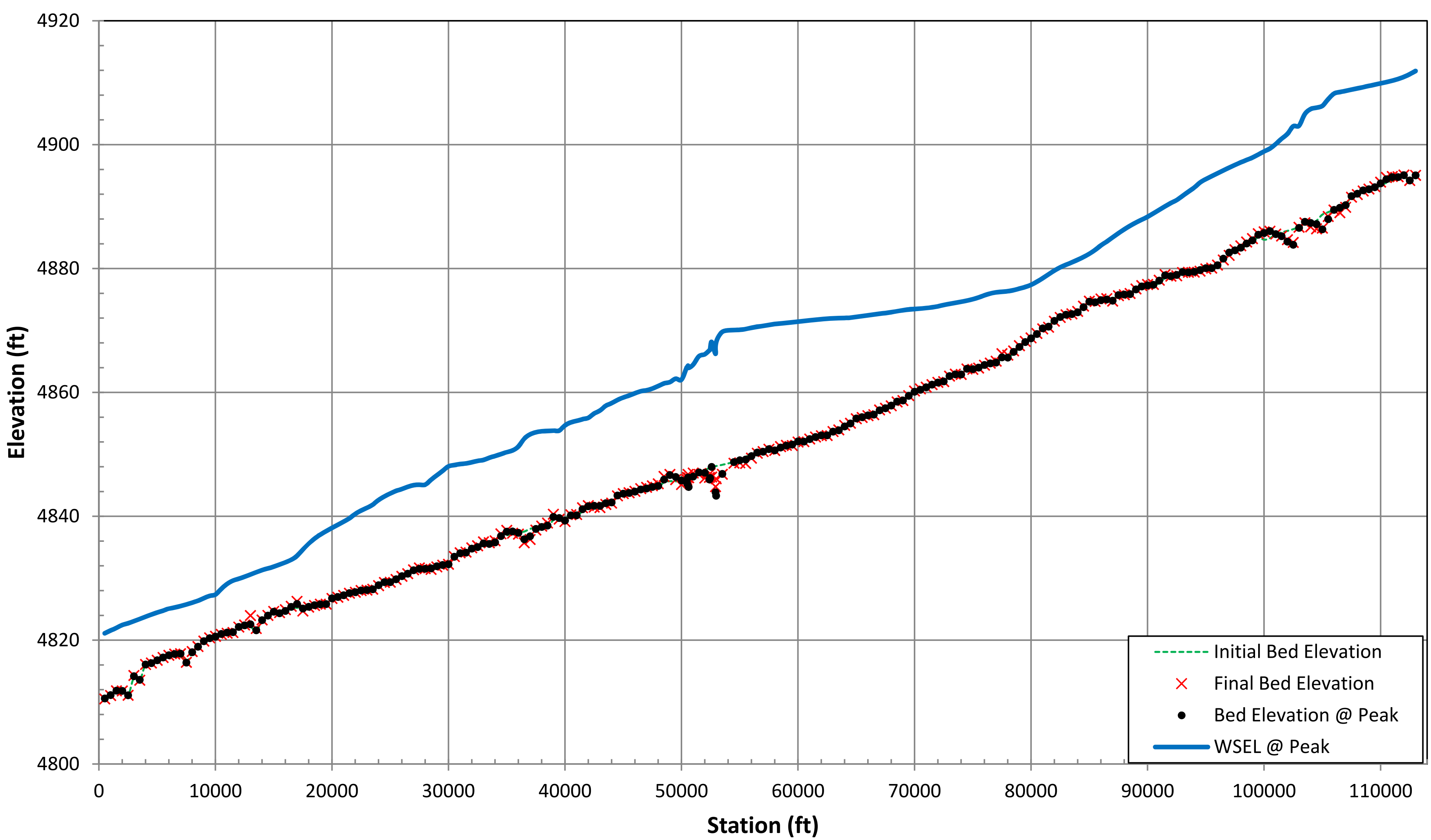
FIGURE 9



Note: LCR sediment transport results from HEC-RAS using n-year hydrographs.

HYDRAULIC AND SEDIMENTATION APPENDIX LITTLE COLORADO RIVER WINSLOW, ARIZONA	**LITTLE COLORADO RIVER BED PROFILES AFTER 1% ACE FLOOD**	U.S. ARMY CORPS OF ENGINEERS LOS ANGELES DISTRICT

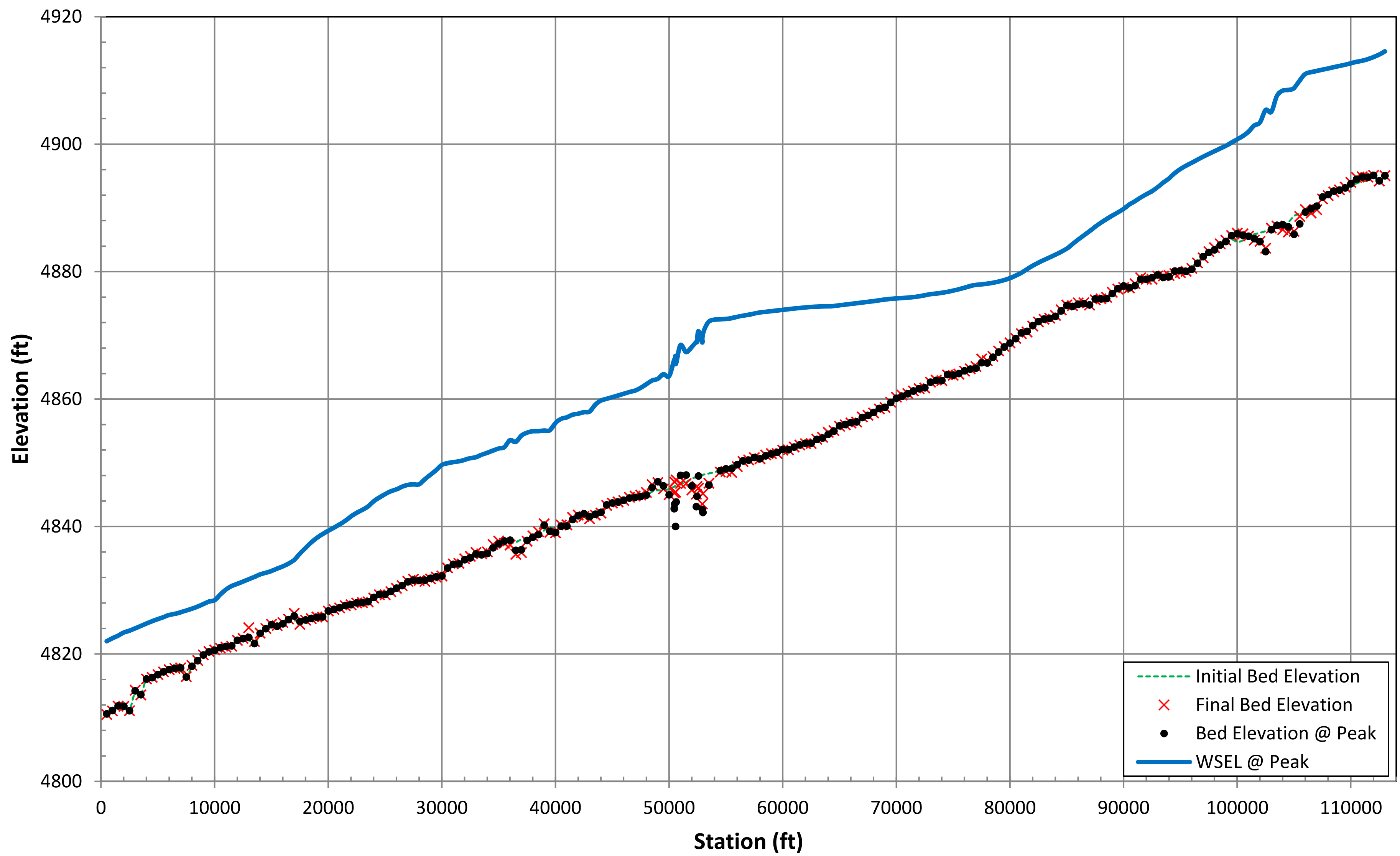
FIGURE 10



Note: LCR sediment transport results from HEC-RAS using n-year hydrographs.

HYDRAULIC AND SEDIMENTATION APPENDIX LITTLE COLORADO RIVER WINSLOW, ARIZONA	**LITTLE COLORADO RIVER BED PROFILES AFTER 0.5% ACE FLOOD**	U.S. ARMY CORPS OF ENGINEERS LOS ANGELES DISTRICT

FIGURE 11



Note: LCR sediment transport results from HEC-RAS using n-year hydrographs.

HYDRAULIC AND SEDIMENTATION APPENDIX LITTLE COLORADO RIVER WINSLOW, ARIZONA	**LITTLE COLORADO RIVER BED PROFILES AFTER 0.2% ACE FLOOD**	U.S. ARMY CORPS OF ENGINEERS LOS ANGELES DISTRICT

FIGURE 12

ATTACHMENTS

LITTLE COLORADO RIVER AT WINSLOW
HYDRAULIC AND SEDIMENTATION APPENDIX
APRIL 2016

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ATTACHMENT 1

FIELD VISIT MEMORANDUM WITH PHOTOS

LITTLE COLORADO RIVER AT WINSLOW
HYDRAULIC AND SEDIMENTATION APPENDIX
APRIL 2016

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CESPL-ED-H 23 August 2011

MEMORANDUM FOR CESPL-ED

SUBJECT: Little Colorado River at Winslow Feasibility Study – Field Visit on 9-11 August 2011

1. The purpose of this memorandum is to document a field visit to Little Colorado River (LCR) in Winslow, Arizona. The purpose of the field visit was to observe the Winslow Levee, the LCR Channel, and Ruby Wash, and to obtain additional bridge/underpass data. Additionally, we visited Homolovi State Park. Messrs. Van Crisostomo, James Chieh, and Adam Bier of Hydraulics Section along with Richard Legere and David Rodriguez from Planning Division visited Winslow from 9 Aug 2011 to 11 Aug 2011.

Homolovi State Park

2. During the field visit to Homolovi State Park on 9 August 2011 at 1600 hours, we met with Richard Lange from the Arizona State Museum. Mr. Lange expressed concerns that changes to the Winslow Levee and LCR could adversely affect the Homolovi I Pueblo. Mr. Lange gave an overview of the Homolovi State Park that included locations of the ancestral Hopi villages that date to the 14th century and also provided tours of two of the village sites referred as Homolovi I and Homolovi II. Homolovi State Park is an economic engine for the Winslow area and is important to the community.

3. Homolovi I is located approximately 150 ft east of the river near Hydrologic Engineering Center – River Analysis System (HEC-RAS) station 390+00 and has seen flood waters encroach on its territory in the past. Figure 1 shows the proximity of LCR as seen from Homolovi I. The village remains mostly covered with sand to help preserve the pueblos from floodwaters; however, Mr. Lange still expressed concern that floodwaters could destroy ancient Hopi pueblos. At Homolovi I, archeologists have left two walls exposed to provide an example of what the village rooms looked like (See Figure 2). The attached photo location map (Enclosure 1) shows where photos were taken in relation to HEC-RAS stations.

4. Homolovi II is located approximately 0.75 miles east of the river near HEC-RAS station 120+00. Homolovi II has seen more extensive excavation due to its location being approximately 100 ft above the river bottom. Figure 3 shows a cluster of rooms from the Homolovi II pueblos While Homolovi II is not threatened by LCR floodwaters, Homolovi I remains near the 100 year floodplain in a location where the river meanders. Alternatives during the F4 phase of the Feasibility Study must consider the effects to the pueblos.

BNSF Railroad Bridge

5. On 10 August 2011, Trent Larson from Navajo County gave a tour of the downstream portion of LCR that affects the city of Winslow and the Winslow Levee system, including the

BNSF Railroad Bridge at HEC-RAS station 529+69. The tour included the following locations: BNSF Railroad Bridge, Route 66 Bridge, the 90 degree bend upstream from the railroad bridge, Winslow Levee, Interstate 40 (I-40) Bridges, and the two locations where LCR impinges on the levee.

6. The BNSF Railroad Bridge was constructed approximately 1000 ft downstream from a 90 degree bend in the river. The bridge is 36 ft wide, spanning approximately 700 ft with five piers. The vegetation in the floodplain upstream and downstream of the bridge consists of dense saltcedar (See Figure 4). According to Table 3-1 in the HEC-RAS Hydraulic Reference Manual, the Manning's n-value for floodplains that have "heavy stand of timber with flow into branches" is estimated to be 0.120 and areas with dense willow have an n-value of 0.150. Averaging the two values, a roughness coefficient of 0.135 will be used for floodplain areas that are populated with saltcedar to account for the dense vegetation. Additionally, the n-value of the channel bed is estimated to be 0.030 based on the channel bed being clean and straight in this section of the river. Furthermore, the upstream and downstream bankfull widths are 155 ft and 185 ft respectively. Figures 5 and 6 show upstream and downstream views of LCR from the railroad bridge.

Route 66 Bridge

7. The Route 66 Bridge is located downstream from the BNSF Railroad at HEC-RAS station 524+51 and was constructed in 2005. The bridge is 45 ft wide, spanning approximately 850 ft with 6 piers. The floodplain upstream from the Route 66 Bridge consists of dense patches of saltcedar (See Figure 7) as well as areas of scattered brush (See Figure 8). The left bank immediately upstream and downstream from the bridge contains scattered brush having an n-value of 0.060. Figure 9 shows a downstream view of the LCR floodplain from the top of the west abutment, which indicates dense patches of saltcedar. The upstream and downstream bankfull widths are approximately 240 and 280 ft respectively. Gabion mesh was used on the abutment slope to protect against erosion under the Route 66 Bridge (See Figure 10).

8. The Cottonwood Wash confluence is approximately 200 ft downstream of the BNSF Railroad Bridge on the right bank of LCR (See Figure 11). Additionally, Figure 12 shows an aerial view of the BNSF Railroad and Route 66 Bridges taken on 28 July 2011. It also shows the patches of saltcedar located upstream and downstream from the bridges.

LCR 90 Degree Bend Upstream from BNSF Railroad Bridge

9. Approximately 1000 ft upstream from the BNSF Railroad Bridge, LCR has a 90 degree bend as it approaches the diversion levee which forces the river to flow north away from the city of Winslow (See Figure 13). Figures 14 and 15 show the downstream and upstream views of the 90 degree bend in LCR respectively as viewed from the diversion levee. The left and right banks contain dense saltcedar with an n-value of 0.135. The channel bottom has an n-value of 0.030.

The outer bank was approximately 10 ft high around the bend with saltcedar populating the floodplain above.

Interstate 40 Bridges

10. Interstate 40 spans LCR with two bridges, one eastbound and one westbound. The eastbound and westbound bridges are located downstream from the Route 66 Bridge at HEC-RAS station 506+10 and 504+94 respectively. Each bridge is 40 ft wide, spanning approximately 1030 ft with 12 piers. Figure 16 shows an aerial view of the I-40 Bridges which shows dense saltcedar upstream (0.135 n-value) and saltcedar downstream with patches of scattered brush (0.060 n-value).

11. Post and wire fencing is located at the toe of the Winslow Levee upstream from the I-40 bridges to protect the levee (See Figure 17). Furthermore, sheet piles were used beneath the I-40 bridges to help protect against erosion (See Figure 18). Debris accumulation was found on two I-40 westbound bridge piers (See Figure 19). Lastly, evidence of erosion was found on the western embankment of the Winslow Levee between the I-40 bridges and the Route 66 Bridge (See Figure 20).

Winslow Levee

12. The 7.2 mile long Winslow Levee was constructed by Navajo County and the Arizona Department of Water Resources between 1986 and 1989. It was designed to contain the 100-year flood of 65,000 cfs. Recent studies indicate that the levee no longer provides 100- year flood protection, and it has been decertified by the Federal Emergency Management Agency, placing approximately 2,700 parcels and 1,500 structures in the floodplain.

13. A lift gate structure is located at approximately HEC-RAS station 435+00 along the Winslow Levee. This structure allows for runoff to flow into the LCR floodplain from the City of Winslow (See Figure 21). The vegetation on the river side of the levee consists mostly of dense saltcedar.

14. A spur dike is located near HEC-RAS station 370+00. The spur dike has riprap protection protecting its riverside slopes, completed in 2009. According to a March 2010 USACE Geotechnical report, the riprap had a 2.5 ft thickness and a toe-down of 6 feet. The spur dike is located at one of the two impingement points and is supposed to divert LCR and prevent it from directly attacking the levee at that location.

15. Debris from the old Winslow Levee was found inside the floodplain at approximately HEC-RAS station 410+00 (See Figure 22). Debris in the floodplain could inhibit the flow of water during a flood event, putting the levee at risk.

Impingement of Winslow Levee

16. Meandering of the Little Colorado River has rerouted flow near the spur dike located near HEC-RAS station 370+00. During March 2008, floodwaters washed approximately 100 ft of the spur dike and impacted the levee bank upstream of the spur dike according to Navajo County. Emergency repairs were completed as concrete riprap was brought in to stabilize the area. Figure 23 shows how LCR is impinging on the Winslow Levee. The vegetation upstream from the spur dike consists of saltcedar on the right bank (when looking downstream) and scattered brush on the floodplain between the levee and LCR (See Figure 24). The left bank downstream from the spur dike is estimated to be 8 ft high with saltcedar on top of the bank along the toe of the Winslow Levee (See Figure 25).

17. Another impingement site is located at approximately HEC-RAS station 260+00 (See Figure 26). At this location, the meandering river has forced the river into a 90 degree angle with the levee and riprap has been placed along this section of the Winslow Levee (See Figure 27). According to the local sponsor, the riprap thickness is 2.5 ft with a toedown of 10 ft. Upstream of the impingement site, the floodplains along both banks contain dense saltcedar (See Figure 28).

18. In 1993, the levee section between the two impingement sites was overtopped. Permanent repairs were completed in December 1994 as riprap was placed by Navajo County on both sides of the levee along this reach (See Figure 29). Additionally in December 2004, piping failure occurred along the same stretch of the levee. Riprap repairs were made in the vicinity of the piping location in 2005 in response to the 2004 piping failure. The emergency repairs were completed by Navajo County.

19. At approximately HEC-RAS station 210+00, car bodies and parts were found along the levee embankment. These car bodies pose a threat to the integrity of the levee (See figure 30).

Ruby Wash Diversion Levee

20. The Ruby Wash Diversion Levee is 5.3 miles long and was designed and constructed by USACE in 1970. Flows in Ruby Wash are diverted east to the Little Colorado River, protecting the Winslow Airport and approximately 500 residents.

21. The Ruby Wash Diversion Levee captures runoff from Ruby Wash guiding it to the confluence with LCR approximately 800 ft upstream from the BNSF Railroad. B ecause Ruby Wash is ephemeral, the channel bed consists of patches of scattered brush, characterized by an n-value of 0.060. The wash also has areas that are clean and straight, which has a roughness coefficient of 0.030 according to Table 3.1 of the HEC-RAS Hydraulic Reference Manual. Figure 31 shows the variation in channel bottom.

Interstate 40, Route 66, and BNSF Railroad Underpasses

22. Interstate 40 has four underpasses that would allow flood waters to get to the City of Winslow. Moving from east to west are Transcon Lane, Oak Street, Ruby Wash, and North Park Drive underpasses. Oak Street is a trapezoidal underpass that is 48 ft wide at the road level. Transcon Lane is also a trapezoidal underpass that is 60 ft wide at the road level. North Park Drive is a rectangular underpass that is 215 ft wide (See Figure 32). Additionally Ruby Wash travels beneath I-40 (approximately 170 ft wide), which could also be an avenue for flood waters to reach the city (See Figure 33).

23. The BNSF Railroad has five underpasses between Hwy 87 and LCR. Each underpass allows for runoff to travel north towards LCR, which could also allow floodwaters to enter more quickly should the Ruby Wash Levee fail. Figure 34 shows the BNSF Railroad Bridge over Ruby Wash, one of the five underpasses. Route 66 also has underpasses at each of these locations.

Conclusions/Recommendations

24. Based on the site visit, the saltcedar and vegetation in the floodplain is denser than previously assumed using aerial photography. The model will be updated using site visit photos taken by Rich Legere and Adam Bier on 9-11 Aug 2011, in addition to photos taken from a helicopter by Planning Division on 28 July 2011.

25. Roughness coefficients were obtained using the recommended values from Arizona Department of Water Resources (ADWR) "Design Manual for Engineering Analysis of Fluvial Systems" as well as Table 3.1 from HEC-RAS Hydraulic Reference Manual. The selected values range from 0.030 to 0.12. The following is a summary of roughness coefficients to be used in hydraulic design.

a. 0.030 – Channel Bed with fine to medium sand
b. 0.060 – Scattered brush, heavy weeds
c. 0.070 – Light brush and trees
d. 0.090 – Residential medium density (Hejl, 1977)
e. 0.12 – Combination of dense willows, summer, straight and heavy stand of timber, little undergrowth with flow into branches (saltcedar)

26. The Winslow Levee between the I-40 Bridge and Route 66 Bridge showed evidence of erosion on the embankment. This stretch of the Winslow Levee is critical due to the proximity to the City of Winslow.

27. The levee has experienced overtopping (1993) and piping (2003) along approximately a 10,000 ft stretch between the two impingement points. This stretch of the Winslow Levee has been reinforced with riprap on both embankments. Furthermore, the two impingement locations

have been reinforced as well, but they still need to be monitored due to the river's proximity to the levee.

28. Any questions should be directed to Mr. Adam Bier of Hydraulics Section at (213) 452-3567 or Adam.J.Bier@usace.army.mil.

Encls

RENE A. VERMEEREN, PE
Chief, Hydrology and Hydraulics Branch

CF: CESPL-PM-C (Kenny)
CESPL-PD-WC (Legere)
CESPL-ED
CESPL-ED-H
CESPL-ED-G
CESPL-ED-D
CESPL-ED-HH (2)

VERMEEREN
CESPL-ED-H

CRISOSTOMO
CESPL-ED-HH

CHIEH
CESPL-ED-HH

BIER
CESPL-ED-HH

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Legend

- PhotoLocations
- Little Colorado River
- LCR Tributaries
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad

N

0 2,500 5,000 10,000 Feet

1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

PHOTO LOCATION MAP
FIELD VISIT
9-11 AUGUST 2011

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

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Figure 1: View from Homolovi I looking west towards LCR (09 Aug 2011)

Figure 2: View from Homolovi I looking northwest towards LCR (09 Aug 2011)

Figure 3: View of pueblos at Homolovi II (09 Aug 2011)

Figure 4: View of saltcedar vegetation upstream from the BNSF Railroad Bridge (10 Aug 2011)

Figure 5: View from left bank near the BNSF Railroad Bridge looking upstream (10 Aug 2011)

Figure 6: View from left bank near the BNSF Railroad Bridge looking downstream (10 Aug 2011)

Figure 7: View from top of west abutment of Route 66 Bridge, looking upstream (10 Aug 2011)

Figure 8: View from left bank near Route 66 Bridge pier looking upstream towards BNSF Bridge (10 Aug 2011)

Figure 9: View from top of west abutment of Route 66 Bridge, looking downstream (10 Aug 2011)

Figure 10: Gabion mesh erosion protection on the western Route 66 Bridge abutment (10 Aug 2011)

Figure 11: Cottonwood Wash confluence with LCR (10 Aug 2011)

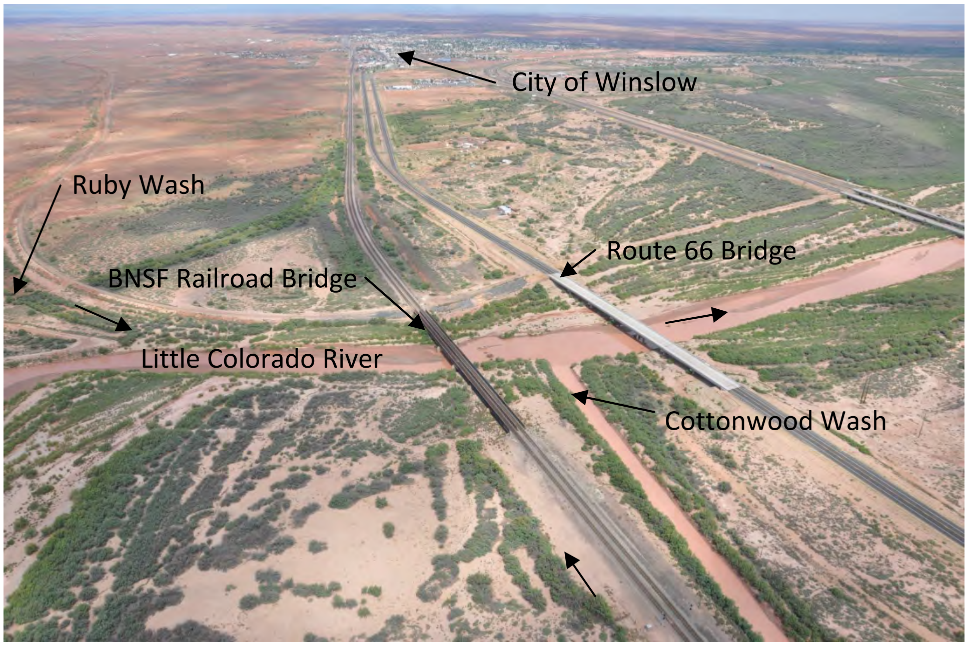

Figure 12: View of LCR, the BNSF and Route 66 Bridges, and Winslow, looking West (28 July 2011)

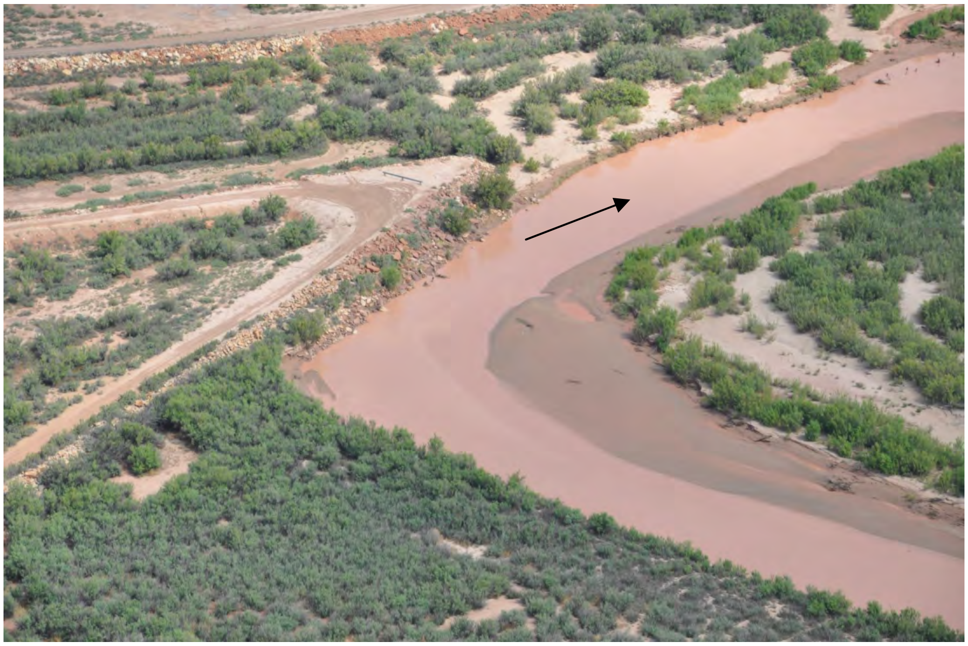

Figure 13: View of 90 degree bend in LCR as it approaches the diversion levee, changing its flow direction from west to north (28 July 2011)

Figure 14: View from diversion dike 1000 ft upstream from BNSF bridge near 90 degree bend in LCR looking downstream (North) towards the BNSF Bridge (10 Aug 2011)

Figure 15: View from diversion dike 1000 ft upstream from BNSF Bridge near 90 degree bend in LCR looking upstream (East) (10 Aug 2011)

Figure 16: View of I-40 bridges with the BNSF and Route 66 bridges further upstream (28 Jul 2011)

Figure 17: View of post and wire fencing along the Winslow Levee toe upstream of the I-40 bridges (10 Aug 2011)

Figure 18: View of sheet piling along the western I-40 bridge abutment (10 Aug 2011)

Figure 19: View of debris accumulation on westbound I-40 Bridge piers, looking downstream (10 Aug 2011)

Figure 20: Evidence of piping was found on the Western embankment of the Winslow Levee between the I-40 bridges and the Route 66 Bridge (10 Aug 2011)

Figure 21: View from Winslow Levee looking East at lift gate structure, HEC-RAS station 43,500 (10 Aug 2011)

Figure 22: Debris inside the floodplain along the Winslow Levee, HEC-RAS station 41,000 (10 Aug 2011)

Figure 23: View of LCR impingement on Winslow Levee and spur dike, looking Southwest (28 July 2011)

Figure 24: View from spur dike near LCR impingement on Winslow Levee looking upstream, HEC-RAS station 37,000 (10 Aug 2011)

Figure 25: View from spur dike near LCR impingement on Winslow Levee looking downstream, HEC-RAS station 37,000 (10 Aug 2011)

Figure 26: View of impingement site looking west, HEC-RAS station 26,000 (28 July 2011)

Figure 27: View from levee looking downstream from impingement site, HEC-RAS station 26,000 (10 Aug 2011)

Figure 28: View from levee looking upstream from impingement site, HEC-RAS station 26,000 (10 Aug 2011)

Figure 29: View of Winslow Levee showing riprap placement typical between HEC-RAS station 37,000 and 26,000 (28 July 2011)

Figure 30: Car body in Winslow Levee embankment (10 Aug 2011)

Figure 31: View of Ruby Wash looking downstream, HEC-RAS station 10,000 (10 Aug 2011)

Figure 32: View of North Park Drive underpass beneath I-40 looking towards Winslow (11 Aug 2011)

Figure 33: View of Ruby Wash underpass beneath I-40 looking North (11 Aug 2011)

Figure 34: View of BNSF Railroad Bridge over Ruby Wash looking South (10 Aug 2011)

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ATTACHMENT 2

SEDIMENT GRADATIONS AND

GRADATION LOCATION MAP

LITTLE COLORADO RIVER AT WINSLOW
HYDRAULIC AND SEDIMENTATION APPENDIX
APRIL 2016

Sieve	Sample #	Sample 4				Sample 6			
No.	size (mm)	Wt. Ret.	% ret	% finer	% in size class	Wt. Ret.	% ret	% finer	% in size class
8	2.38	0	0	100	0.0	0	0	100	0.0
10	2	0	0	100	0.0	0	0	100	0.0
16	1.19	0	0	100	0.0	0	0	100	0.0
30	0.59	0.6	0.075	99.925	0.1	0	0	100	0.0
40	0.42	4	0.573	99.427	0.5	0	0	100	0.0
50	0.297	36.4	5.108	94.892	4.5	1.3	0.184	99.816	0.2
100	0.149	532.6	71.459	28.541	66.4	426	60.636	39.364	60.5
200	0.074	222.9	99.228	0.772	27.8	270.4	99.007	0.993	38.4
	Pan	6.2	100	0	0.8	7	100	0	1.0
		802.7			100	704.7			100

d16	d50	d84
0.109	0.186	0.265

d16	d50	d84
0.097	0.168	0.248

Sieve	Sample #	Sample 7				Sample 9			
No.	size (mm)	Wt. Ret.	% ret	% finer	% in size class	Wt. Ret.	% ret	% finer	% in size class
8	2.38	0	0	100	0.0	0	0	100	0.0
10	2	0	0	100	0.0	0	0	100	0.0
16	1.19	0	0	100	0.0	0	0	100	0.0
30	0.59	0	0	100	0.0	0.3	0.044	99.956	0.0
40	0.42	1.7	0.250	99.750	0.2	6	0.929	99.071	0.9
50	0.297	60.4	9.123	90.877	8.9	61.4	9.984	90.016	9.1
100	0.149	549.1	89.790	10.210	80.7	342.2	60.448	39.552	50.5
200	0.074	68.1	99.794	0.206	10.0	255.6	98.142	1.858	37.7
	Pan	1.4	100	0	0.2	12.6	100	0	1.9
		680.7			100	678.1			100

d16	d50	d84
0.157	0.209	0.280

d16	d50	d84
0.096	0.172	0.274

NOTE: Soil samples from the Winslow area received from Tom Hieb, Bureau of Reclamation, 2-25-02

Sieve	Sample #	Sample 10				Sample 12			
No.	size (mm)	Wt. Ret.	% ret	% finer	% in size class	Wt. Ret.	% ret	% finer	% in size class
8	2.38	0	0	100	0.0	0	0	100	0.0
10	2	0	0	100	0.0	0	0	100	0.0
16	1.19	0.1	0.016	99.984	0.0	0	0	100	0.0
30	0.59	7.9	1.273	98.727	1.3	0.2	0.122	99.878	0.1
40	0.42	78.8	13.811	86.189	12.5	0.6	0.486	99.514	0.4
50	0.297	210.5	47.303	52.697	33.5	1	1.094	98.906	0.6
100	0.149	279.7	91.806	8.194	44.5	37.3	23.755	76.245	22.7
200	0.074	50	99.761	0.239	8.0	119.3	96.233	3.767	72.5
	Pan	1.5	100	0	0.2	6.2	100	0	3.8
		628.5			100	164.6			100

d16	d50	d84
0.296	0.285	0.396

d16	d50	d84
0.083	0.116	0.189

NOTE: Soil samples in the Winslow area received from Tom Hieb, Bureau of Reclamation, 2-25-02

ATTACHMENT 2
LITTLE COLORADO RIVER
GRADATION DATA

		Sample #18				Sample #19			
sieve no.	size (mm)	Wt. Ret. (gm)	% ret	% finer	% in size class	Wt. Ret. (gm)	% ret	% finer	% in size class
1/2	13	0	0	100	0.0	0	0	100	0.0
3/8	9.53	0	0	100	0.0	0	0	100	0.0
1/4	6.4	0	0	100	0.0	0	0	100	0.0
8	2.4	1	0.234	99.766	0.2	0	0	100	0.0
10	2	0	0.234	99.766	0.0	0	0	100	0.0
16	1.2	0	0.234	99.766	0.0	0	0	100	0.0
30	0.6	0	0.234	99.766	0.0	3	0.683	99.317	0.7
40	0.4	0	0.234	99.766	0.0	15	4.100	95.900	3.4
50	0.3	11	2.810	97.190	2.6	79	22.096	77.904	18.0
100	0.149	362	87.588	12.412	84.8	295	89.294	10.706	67.2
200	0.07	53	100	0	12.4	47	100	0	10.7
		427			100	439			100

d16	d50	d84	dg	d16	d50	d84	dg
0.087	0.190	0.261	1.736	0.126	0.213	0.325	1.604

		Sample #21				Sample #25			
sieve no.	size (mm)	Wt. Ret. (gm)	% ret	% finer	% in size class	Wt. Ret. (gm)	% ret	% finer	% in size class
1/2	13	0	0	100	0.0	0	0	100	0.0
3/8	9.53	0	0	100	0.0	0	0	100	0.0
1/4	6.4	0	0	100	0.0	0	0	100	0.0
8	2.4	0	0	100	0.0	0	0	100	0.0
10	2	0	0	100	0.0	0	0	100	0.0
16	1.2	0	0	100	0.0	0	0	100	0.0
30	0.6	1	0.321	99.679	0.3	1	0.260	99.740	0.3
40	0.4	14	4.808	95.192	4.5	20	5.455	94.545	5.2
50	0.3	110	40.064	59.936	35.3	104	32.468	67.532	27.0
100	0.149	176	96.474	3.526	56.4	206	85.974	14.026	53.5
200	0.07	11	100	0	3.5	54	100	0	14.0
		312			100	385			100

d16	d50	d84	dg	d16	d50	d84	dg
0.170	0.260	0.375	1.485	0.138	0.231	0.364	1.623

NOTE: Data from Bureau of Reclamation Sediment Study, May 2003

ATTACHMENT 2
LITTLE COLORADO RIVER GRADATION DATA

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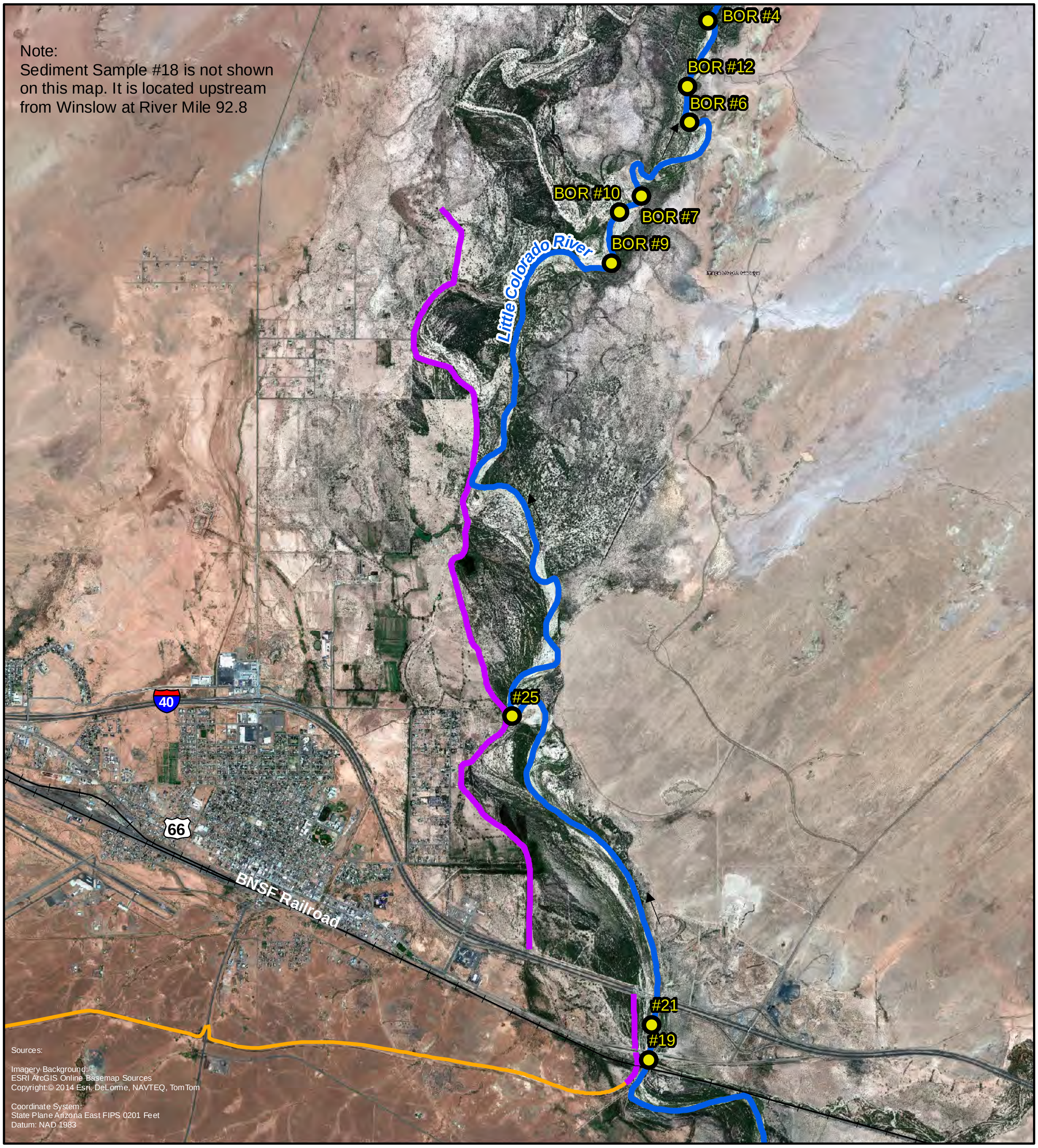

Legend

- Sediment Sample Location
- Little Colorado River
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad

N

0 2,500 5,000 10,000 Feet

1 in = 5,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

SEDIMENT SAMPLE LOCATION MAP

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

ENCLOSURE 2-1

ATTACHMENT 3

CHANNEL GEOMORPHOLOGY

LITTLE COLORADO RIVER AT WINSLOW
HYDRAULIC AND SEDIMENTATION APPENDIX
APRIL 2016

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CESPL-ED-HH 3 September 2013

MEMORANDUM FOR RECORD

SUBJECT: Little Colorado River at Winslow –River Geomorphology

1. References:

- U.S. Bureau of Reclamation, *Analysis of Little Colorado River Stability between Holbrook and Winslow, Arizona, Report of Findings, Little Colorado River Sediment Study*, May 2003.

Purpose

2. The purpose of this memorandum is to document the geomorphic analysis that was conducted along the Little Colorado River between Holbrook and Winslow. A major benefit of conducting a geomorphic analysis is to provide a broad perspective on the long–term behavior of the Little Colorado River between Holbrook and Winslow, particularly in regards to the extent of aggradation and/or degradation in this reach.

Geomorphology Overview

3. The United States Bureau of Reclamation (USBR) conducted a sediment study for the LCR in the Winslow area and presented a report in 2003 entitled, "*Analysis of Little Colorado River Stability between Holbrook and Winslow, Arizona, Report of Findings, Little Colorado River Sediment Study, May 23, 2003*".

4. The alluvial units mapped along the Little Colorado River are primarily delineated on the basis of their geomorphic characteristics. These characteristics include the elevation and relative position of each unit to the active channel (Qac) and adjacent map units, surface morphology, and the dominant type and relative coverage of vegetation on the surface. These types of indicators are widely used.

- Unit Qac – active channel – primarily silty sand alluvium with clay–rich alluvium in meander bends and backwater channels.
- Unit Qa1, Qa1a, Qa1b – Desert Broom terrace – sandy alluvium that forms low point bars and floodplains immediately adjacent to the active channel with either no vegetation or sparse young Tamarisk and Desert Broom.
- Unit Qa2, Qa2a, Qa2b – Tamarisk terrace – silty sand alluvium covered by thick vegetation, primarily Tamarisk.
- Unit Qa4 – Moenkopi terrace – dark red clay and silt–rich alluvium that forms the highest terrace associated with the Little Colorado River.

- Unit Qe – Dunes – fine to medium–grained eolian sand.
- Unit Qpc – undifferentiated paleochannels – numerous meander scars and recently abandoned channels.

Geomorphology Mapping

5. The geomorphology of the Little Colorado River in the Winslow area differs significantly compared to the Holbrook reach. Enclosure 3-1 shows the Winslow Geomorphic Map. In the Winslow reach, the width of the floodplain increases dramatically. It appears that this change in the floodplain width is related to the increase in basin area, and hence, a related increase in stream flow immediately downstream of the confluences of Chevelon Canyon, Clear Canyon, Cottonwood Creek, and Jacks Canyon with the Little Colorado River. Although the floodplain near Winslow is much wider than upstream reaches on the Little Colorado River, the Winslow Levee cuts off the majority of additional flood plain.

6. Geomorphic mapping for this study was limited to the river and terraces within the levee. The Little Colorado River near the Homolovi I Pueblo is bounded on the right bank by bedrock and on the left bank by the Winslow Levee. The Moenkopi terrace (Qa4) is present along the right bank adjacent to bedrock, but not along the left bank within the levee. Terraces within the levee are limited to primarily the Tamarisk and Desert Broom alluvium. The Cottonwood terrace is outside the levee.

7. Dunes (Qe) in the area are quite extensive and exist on both the east and west sides of the river. Mature Tamarisk or Cottonwood trees (50–100 years old) stabilize many of the dunes. Smaller dunes are also present on the younger Desert Broom and Tamarisk terraces.

8. Near the Homolovi I Pueblo, channel dredging and channelization between 1984 and 1993 shifted the channel to the east from a position against the levee, indicated by the Qa1a channel.

9. The previous channel, unit Qa1a, is now only accessed during larger flows. Much of the active channel through this reach has formed meanders following dredging, and has migrated to its easternmost extent near the Homolovi I Pueblo where it flows against bedrock. Large dune complexes prevalent along the east side of the river are sparsely vegetated, modified by high flows (unit Qa2b) on the Little Colorado River.

10. Dunes on the west side of the river are more heavily vegetated with Tamarisk and Cottonwood. Further north along the Winslow Levee, a broad Tamarisk terrace (Qa2) is present adjacent to the river. Channel splays on the right bank apparently associated with the flooding in 1993 appear to be more significant in this area than on other Qa2 surfaces.

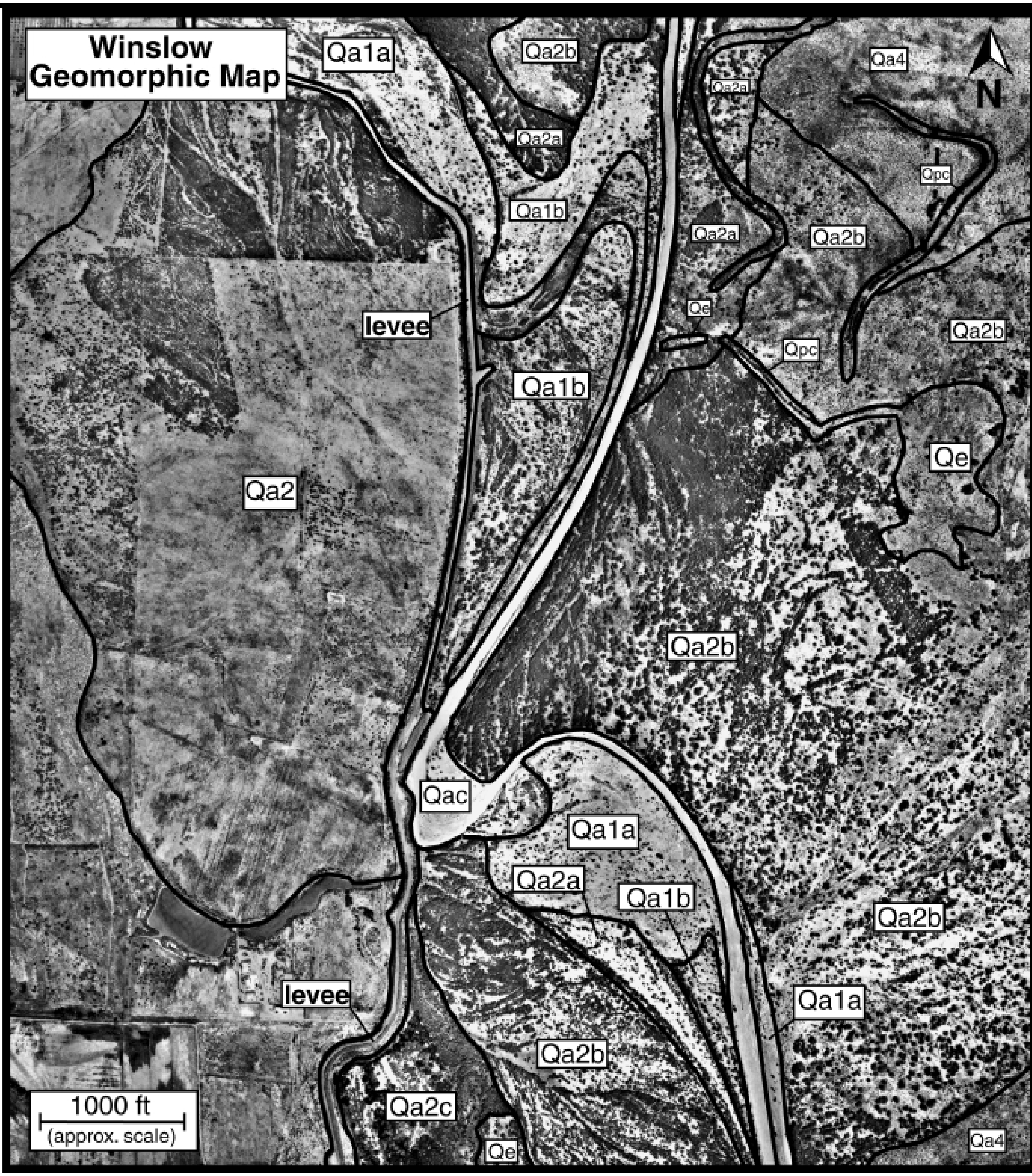
11. The north bank of the river, in the west verging meander cut into the older Tamarisk alluvium (unit Qa2b), is also significantly higher than bank cuts in many other Qa2 surfaces in the Holbrook–Winslow reach. This high bank and densely vegetated surface on the Qa2b terrace appear to be factors in maintaining the accentuated meander at this location. The Qa2b surface in this area appears to grade to the Moenkopi terrace to the northeast with

distance from the active channel. The behavior of the Little Colorado River in this reach is similar to that observed in other reaches, in that the river is incising older alluvium and migrating across a wide flood plain. The gradual transition in elevation from the surface of the younger Tamarisk alluvium (Qa2) to the Moenkopi terrace suggests that the river has migrated across a much wider floodplain at this site. Other characteristics that are unusual in this reach, when compared to other reaches, and indicate that the reach has been highly modified, include the narrow width of the active channel and the extreme height of the Qa2b surface above the active channel.

12. Questions on this matter should be directed to Mr. James Chieh at (213) 452-3571.

Encl

James Chieh, P.E.
Senior Hydraulic Engineer,
Hydraulics Section



Source: USBR 2003 Study

LITTLE COLORADO RIVER, WINSLOW, AZ FEASIBILITY STUDY	**ENCLOSURE 3-1 LITTLE COLORADO RIVER GEOMORPHIC MAP**	U.S. ARMY CORPS OF ENGINEERS LOS ANGELES DISTRICT

ATTACHMENT 4

GEOTECHNICAL EVALUATION OF LEVEE FRAGILITY

LITTLE COLORADO RIVER AT WINSLOW
HYDRAULIC AND SEDIMENTATION APPENDIX
APRIL 2016

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US Army Corps of Engineers

Los Angeles District & San Francisco District
Geotechnical Branches

Geotechnical Evaluation of Levee Fragility: Winslow Levee and Ruby Wash Diversion Levee, LCR at Winslow Feasibility Study, Winslow, AZ

left, Winslow Levee, looking north (downstream) at fragility-curve modeled HEC-RAS Winslow Station 37000 (which is in center of frame). Photograph by US Army Corps of Engineers, 18 October 2011.

by
US Army Corps of Engineers
Geotechnical Branches in Los Angeles District and San Francisco District
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23 May 2012

Geotechnical Evaluation of Levee Fragility: Winslow Levee and Ruby Wash Diversion Levee, LCR at Winslow Feasibility Study, Winslow, AZ

Report Prepared by:

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Geotechnical Branch, Engineering Division, Los Angeles District, US Army Corps of Engineers

Table of Contents:

1.0 Purpose

This report was prepared by the US Army Corps of Engineers ("Corps"), Los Angeles District. The purpose of the report is to evaluate the expected geotechnical performance of the Winslow and Ruby Wash Diversion levees, located near Winslow, Arizona. This evaluation is intended for use in performing an economic cost/benefit analysis to determine if a flood damage reduction project is feasible in accordance with Corps of Engineers Manual EM 1110-2-1619, Risk-Based Analysis for Flood Damage Reduction Studies.

This analysis is intended only for feasibility-level analysis. The values presented in this report are not design values for new structures, and appropriate exploration, lab testing and engineering analysis should be performed for new project design.

2.0 Personnel

Corps personnel that performed the analysis and prepared this work were:

Mark Chatman, P.G.	Chief Geologist, Los Angeles District
Stephen Brown, E.I.T.	Civil Engineer, Los Angeles District
Brian Hubel, P.E., G.E.	Geotechnical Engineer, San Francisco District

3.0 Project Description

The project is located in Winslow, Arizona, approximately 55 miles east of the intersection of Highway 17 and Highway 40 in the north eastern portion of Arizona, where the Little Colorado River and Ruby Wash meet. Figure 1 is a project vicinity map. The Little Colorado River (LCR) generally runs from south to north near Winslow. Ruby Wash joins the LCR just south of State Route 87. The Ruby Wash Diversion Levee generally runs in an east-west direction. Figure 2 is an aerial photograph of the project location identifying relevant landmarks. The western portion of the Ruby Wash Diversion Levee is part of the system, but is in the process of FEMA accreditation, and is not addressed in this report. It is assumed to be stronger than the portions of the levee to be evaluated herein. The initial FEMA application for accreditation of the western portion was rejected, although the sponsor has indicated that with some additional engineering documentation support, it is expected that the accreditation will be approved. The division of the western and eastern portions of the levee occurs at an internal ridge of high ground that divides shallow flood basins.

The project is intended to reduce flooding in Winslow from high flows along the LCR and Ruby Wash. The existing levees have a long history of varied construction, flooding damage, repairs and continual improvements that have resulted in the levee configuration as it exists today. The Los Angeles District Geotechnical Branch prepared a comprehensive literature review of the levee history which is presented in a 12 March 2010 report, titled "Summary of Winslow Levee and Ruby Wash Diversion Levee, Winslow AZ (Navajo County): history, composition, foundation)." This report was relied upon heavily in preparation of this analysis and report, and should be reviewed for details not included in this report.

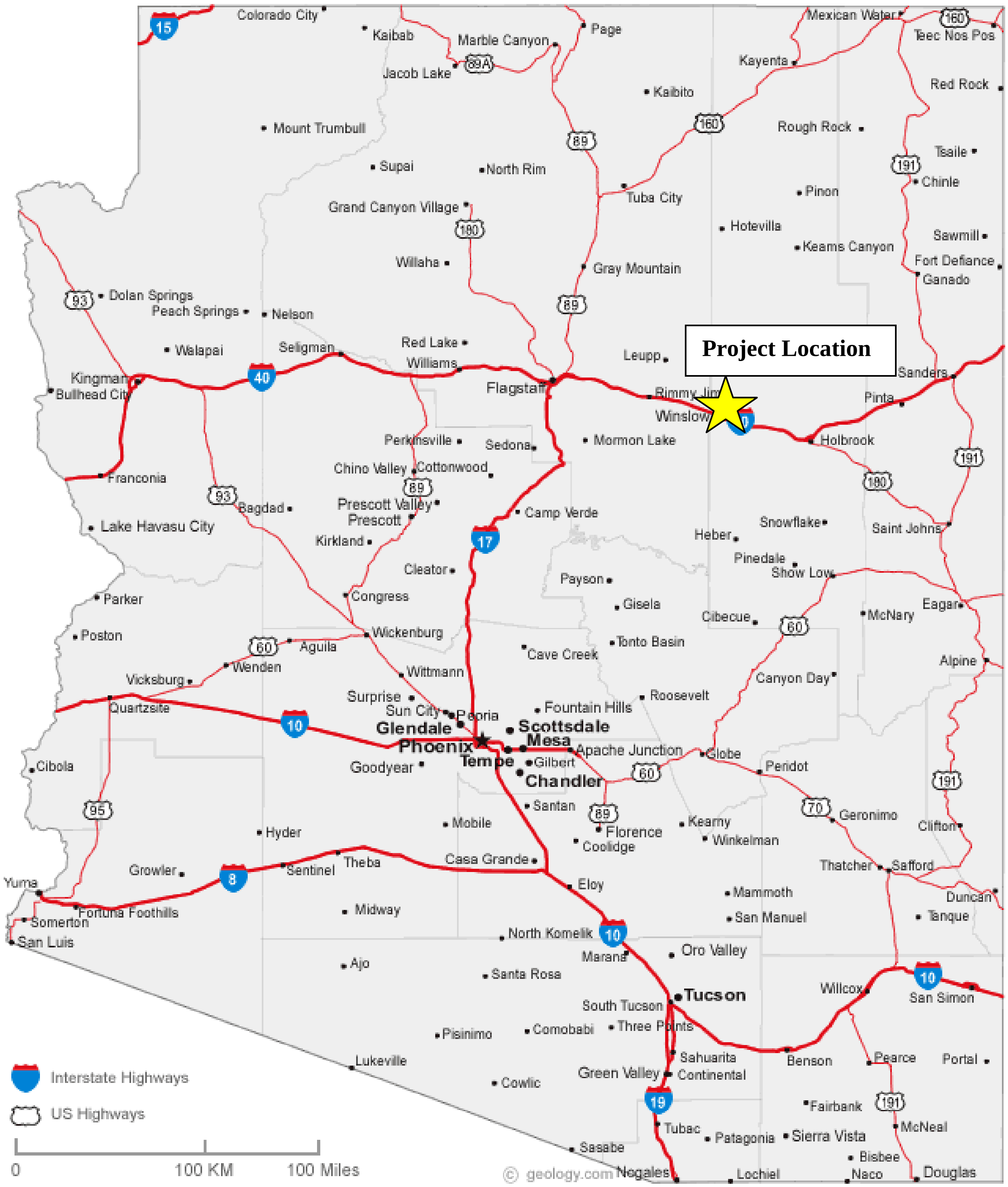


Figure 1. Project Vicinity Map

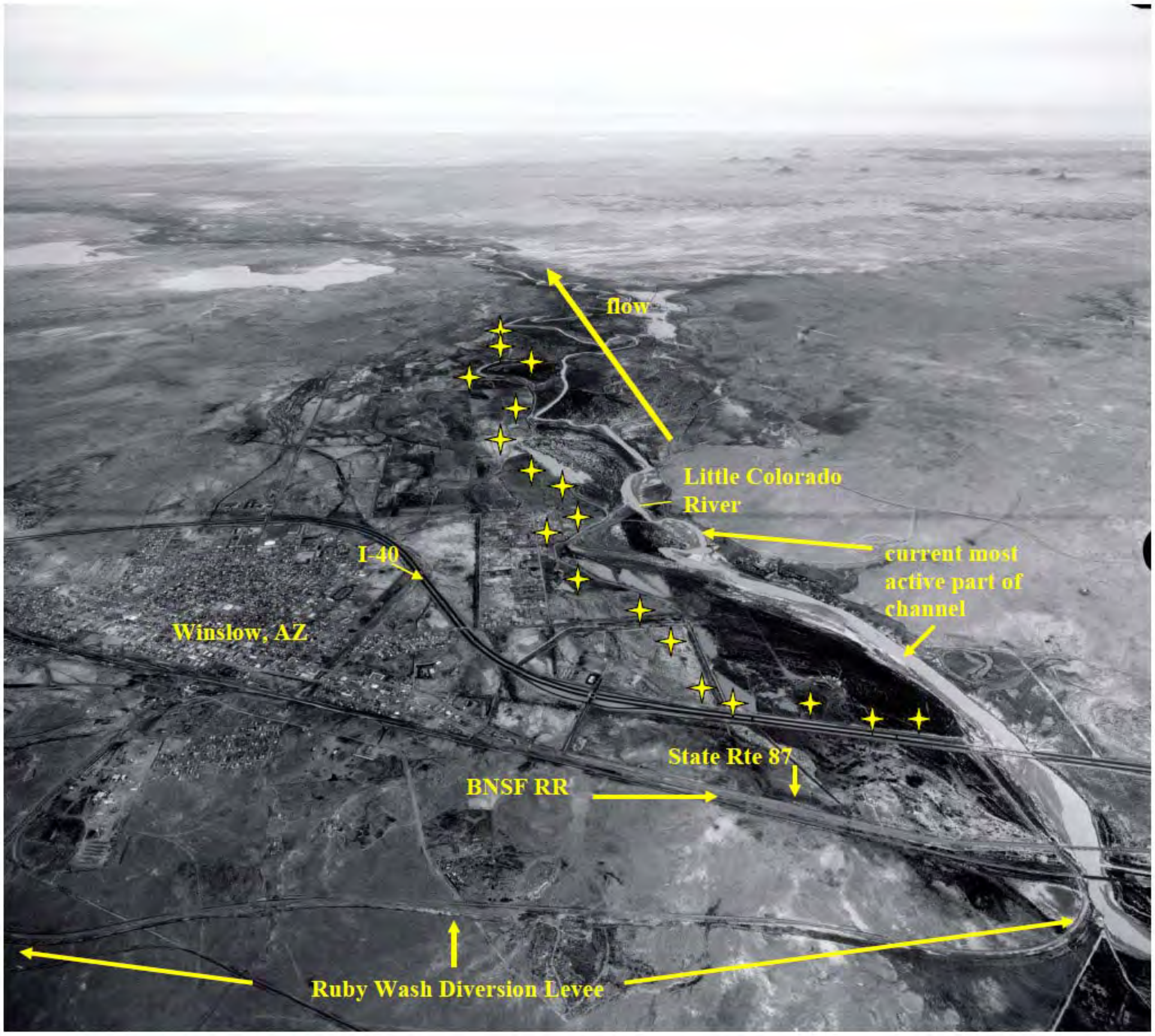


Figure 2. Aerial Photograph of Project (stars mark approximate Winslow Levee alignment)

4.0 Scope of Work, Key Definitions and Assumptions.

The scope of work completed to prepare this report included:

- literature review of geological, geotechnical, design, construction, and performance information,
- a site visit by the geotechnical and geology team to gain an understanding of the project condition, function, design, construction, and project consequences,
- selection of levee "index points" that were judged to be critical in evaluation of flood risk for the project based on a review of levee geometry, hydraulic loading, geotechnical and geologic, conditions, potential economic consequences and performance history,
- engineering analysis including seepage, slope stability, erosion and miscellaneous failure modes to develop fragility curves that describe the probability of unsatisfactory performance ($P_{(u)}$) of the levee as a function of river stage elevation, and

- preparation of this technical report.

As discussed in ETL 1110-2-556 probabilistic engineering analysis is a complex and immature field in geotechnical engineering, and the results of this analysis should be used and interpreted with care.

From ETL 1110-2-556:

> *The application of probabilistic analysis in geotechnical engineering and other areas of civil engineering is still an emerging technology. Much experience with such procedures remains to be gained, and the appropriate form and shape of probability distributions for the relevant parameters are not known with certainty. The methods described herein should not be expected to provide "true" or "absolute" probability-of-failure values but can provide consistent measures of relative reliability when reasonable assumptions are employed. Such comparative measures can be used to indicate, for example, which reach (or length) of levee, which typical section, or which alternative design may be more reliable than another. They also can be used to determine which of several performance modes (seepage, slope stability, etc.) governs the reliability of a particular levee.*

The primary goal of a flood damage reduction feasibility study is to determine the cost/benefit ratios to evaluate if a new flood damage reduction project is warranted. Traditionally, when a levee was "certified" it meant that stability factors of safety and other criteria met recommended design minimums and the levee was assumed to hold (without breach) to the levee crest. If the levee was "not certified" the levee was assumed not to exist and the flood plains were mapped accordingly. This is quite unrealistic, as the levee provides some economic benefits, even if it is a weak or fragile levee.

Corps planning has adopted a risk and uncertainty modeling approach, requiring that that the geotechnical performance of the levee be considered when determining cost/benefit ratios. The geotechnical performance is stochastically incorporated into the economics by the use of levee fragility curves that express the probability that the levee will have unsatisfactory performance for a given river stage. Typically, fragility curves are used in the economic FDA program in a joint probability approach combining event frequency and probability of unsatisfactory performance, such that the damages for a given event are effectively scaled by the probability of unsatisfactory performance. The damages for all possible events are determined and annualized to compute estimated annual damages. For some complicated projects the curves may be used directly in the H&H modeling.

Probability of Unsatisfactory Performance ($P_{(u)}$) is used to define fragility curves. $P_{(u)}$ does not directly describe the probability that the levee will catastrophically fail under a given load, but rather describes the probability that ground conditions exist that would result in a limit state (factor of safety =1.0) being exceeded under the given load for a certain set of assumptions.

Important assumptions for the work performed in this analysis include:

- Steady-state seepage is reached in both the seepage and stability analyses, meaning the water surface in the river is constant for a sufficient period of time such that the phreatic surface across the levee is fully developed.

- A slope stability failure surface with a factor of safety <1.0 has to be sufficiently large such that a majority of the levee crest is included in the failure mass. The criterion for unsatisfactory performance is that the failure surface must include most of the crest. If less than 10 feet remains outside the failure plane, the failure surface is sufficiently large to constitute unsatisfactory performance.
- Similar to the slope stability analysis, erosion performance was defined to be unsatisfactory if erosion progression resulted in a crest width less than 10 feet for the given loading.
- Seepage was determined to be unsatisfactory if a vertical exit gradient at the landside toe exceeded the critical gradient of the soil.
- All uncertainty in the levee performance is aleatory, meaning the calculation and modeling methods are assumed to be accurate, and that all uncertainty is in our knowledge of the ground conditions.
- Soil profiles and property distributions represent the conditions in the field. Significant interpretation and judgment was used to statistically describe the soil conditions for the project. Statistical descriptions generally include our best estimate of a soil parameter and standard deviation around the expected value that describe the likelihood of what the permeability of the layer is. Exploration data is widely spaced, and may not represent the conditions at all locations. Additionally, laboratory test data was limited, especially related to engineering properties of shear strength and permeability which are important parameters in the analysis. The analysis performed included a wide distribution of possible soil properties in an attempt to incorporate the uncertainty in the data.

As noted in the assumptions, there is very little laboratory test data available for development engineering parameters. The intent of this work is to support the Feasibility Scoping Meeting. If the feasibility study continues beyond the Feasibility Scoping Meeting, additional engineering exploration and laboratory testing should be performed. This information and data will serve as additional support and justification for selected engineering parameters in the final cost/benefit determination. Due to the current lack of lab testing data, the uncertainties presented in this report are very large. Additional exploration and lab testing could result in a different understanding of the ground conditions, and could change understanding of the levee fragility.

5.0 Geology and Seismicity[1]

The site is located in the Colorado Plateau physiographic province, which is one of three physiographic and structural provinces in the State of Arizona: the Colorado Plateau, Transition Zone, and Basin and Range. The Colorado Plateau is generally characterized by broad, relatively flat-lying, seismically stable mesas. Geologic mapping of the project area by Richards, Reynolds, Spencer and Pearthree (2000) is shown in Figure 3.

[1] This discussion has partly been copied from Kleinfelder (2009)

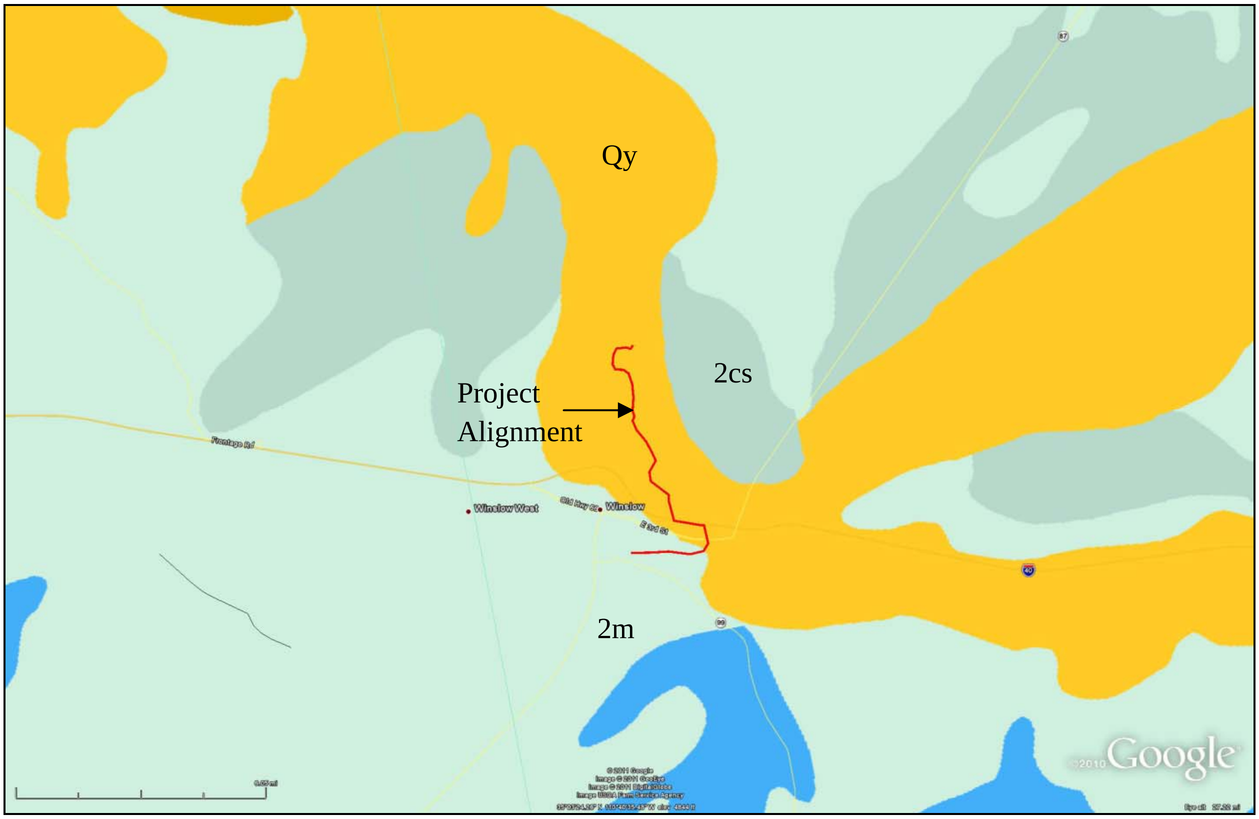


Figure 3. Geologic Map

Mapping indicates Triassic Moenkopi Formation (2m - Moenkopi) unit exposed at the surface at the site near the Ruby Wash end of the project. The Moenkopi Formation was deposited in a near-shore, braided stream environment during the mid to upper Triassic time period (230 – 245 million years ago). The Moenkopi is generally red in color with interbedded and laterally discontinuous claystone/siltstone lenses. Gypsum and other evaporite minerals intermittently occur as thin laminations between the claystone/siltstone interbeds. In general, weathering of this rock unit has produced a relatively thin residual cover of silty and clayey sand. The Moenkopi was observed in outcroppings throughout the project area, especially within washes and the low flow channel excavation. These intermittent washes are generally subject to flash-flood conditions during infrequent rain events.

Mesas far outside the LCR river banks, on both sides of the river, expose the Shinarump Conglomerate Member of the Chinle Formation (Late Triassic) (2cs). This formation is generally described as basalt conglomerate and pebbly sandstone of the Chinle Formation is relatively resistant to erosion and forms extensive benches in some parts of the Colorado Plateau. (210-230 Ma {million years ago}).

In the Winslow Levee portion of the project, surface geology is generally mapped as Holocene surficial deposits (Qy), described as unconsolidated deposits associated with modern fluvial systems. This unit consists primarily of fine-grained, well-sorted sediment on alluvial plains, but also includes gravelly channel, terrace, and alluvial fan deposits on middle and upper piedmonts (0-10 ka {thousand years ago}). More details regarding the depositional morphology of the Qy can be found in the 2010 Los Angeles District literature review report. In general, the deposits are alluvial sandy and silty soils with some fine grain deposits and some Eolian deposits. Details of the foundation conditions at each index point are discussed in the Selection of Index Points section of the report.

Winslow, Arizona has relatively low seismicity. United States Geological Survey (USGS) mapping date 2008 indicates that a peak horizontal ground acceleration (PGA) of approximately 0.04 g has a 10 percent chance of exceedance in 50 years and that a PGA of about 0.11 g has a 2 percent chance of exceedance in 50 years. Deaggregation of the seismic hazard indicates that earthquakes of magnitude 5 to 6 Mw within 10 to 25 km of the site contribute most significantly to the seismic hazard at the mean return interval of 2475 years. These relatively small accelerations and magnitudes are also judged unlikely to cause seismically induced liquefaction or large slope deformations. Due to the extremely remote joint probability of an earthquake and flood simultaneously impacting the levee, seismic failure modes were not considered in the fragility analysis of the Winslow levees.

6.0 Selection of Index Points

The levee, for the purposes of this analysis, was divided into reaches. A reach was defined as a segment of levee which, if a breach were to occur at any point within that segment, would likely result in similar damages. An index point was defined as a critical cross section at a specific station within each reach. The project geotechnical team considered levee geometry, geotechnical conditions, hydraulic loading, past performance and potential economic consequences in selecting index points for levee fragility monitoring. If conditions did not readily allow for determining between two locations in a reach, which was likely to have worse geotechnical performance, both were evaluated. The most fragile index point was chosen to represent the levee in that reach. Three reaches were defined and four index points were selected for geotechnical fragility evaluation. The approximate location of these points is shown on Figure 4. A brief description of each index point, including the rationale for selection, and the engineering properties are discussed in sections 6.1 to 6.4.

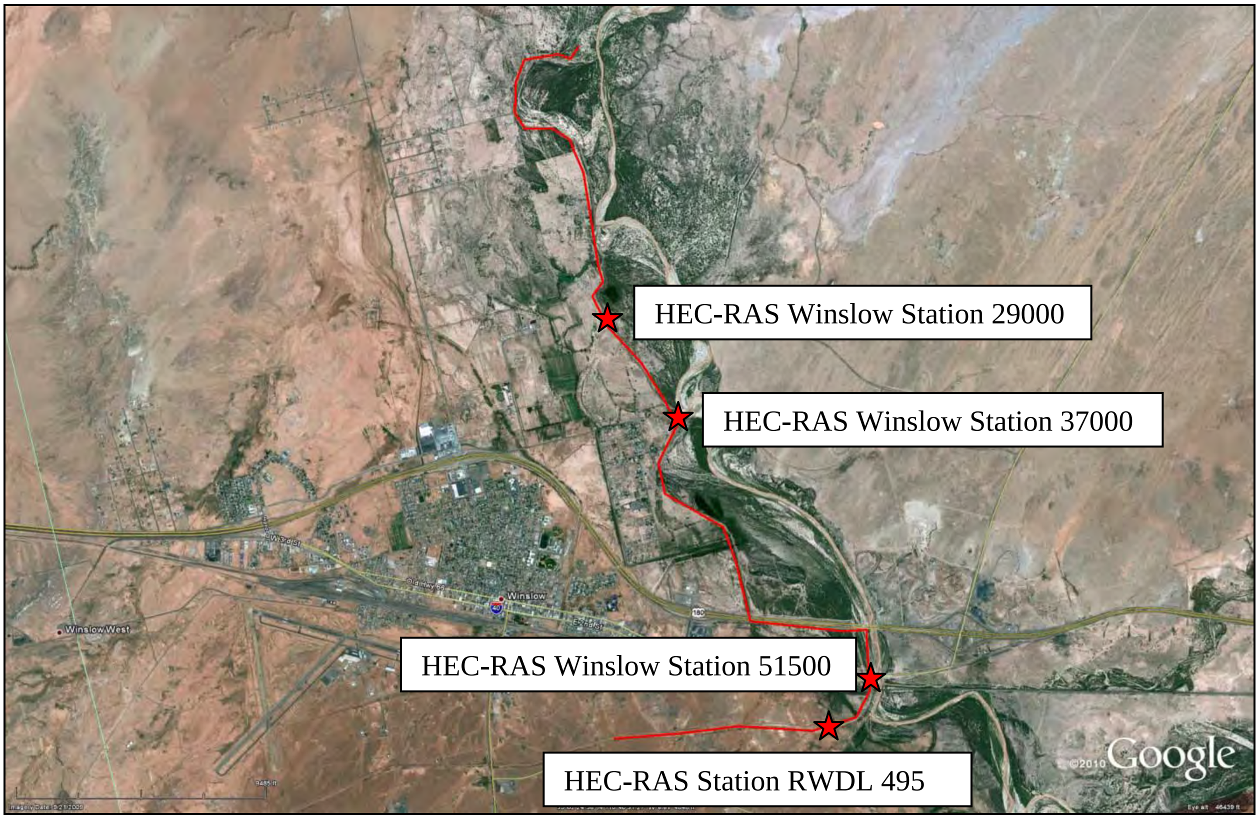


Figure 4. Index Point Locations

6.1 HEC-RAS Station RWDL 495

HEC-RAS Station RWDL 495 was selected as an index point to evaluate levee fragility for the segment of levee between the levee RWDL segment that is being considered for accreditation (evaluation by others) and where the Ruby Wash Diversion Levee joins the Winslow levee at the LCR (near the railroad and highway bridges). This was chosen as an index point because the levee is narrow, has a lower crest elevation, and does not have landside slope armoring and has a nearby levee penetration. In addition, the foundation conditions in the general vicinity consist of alluvium to varying depths overlying the Moenkopi Sandstone foundation.

6.1.1 Hydraulic Loading and Consequences

Hydraulic loading provided by the Los Angeles District H&H Section indicates that RWDL 495 will be overtopped with a return period between 50 and 100 years. A levee breach, if it occurred at this location, would result in flooding of the City of Winslow. Figure 5 illustrates the general route of flooding. Table 1 summarizes the hydraulic loading for RWDL 495 as provided by the Los Angeles District H&H Section.

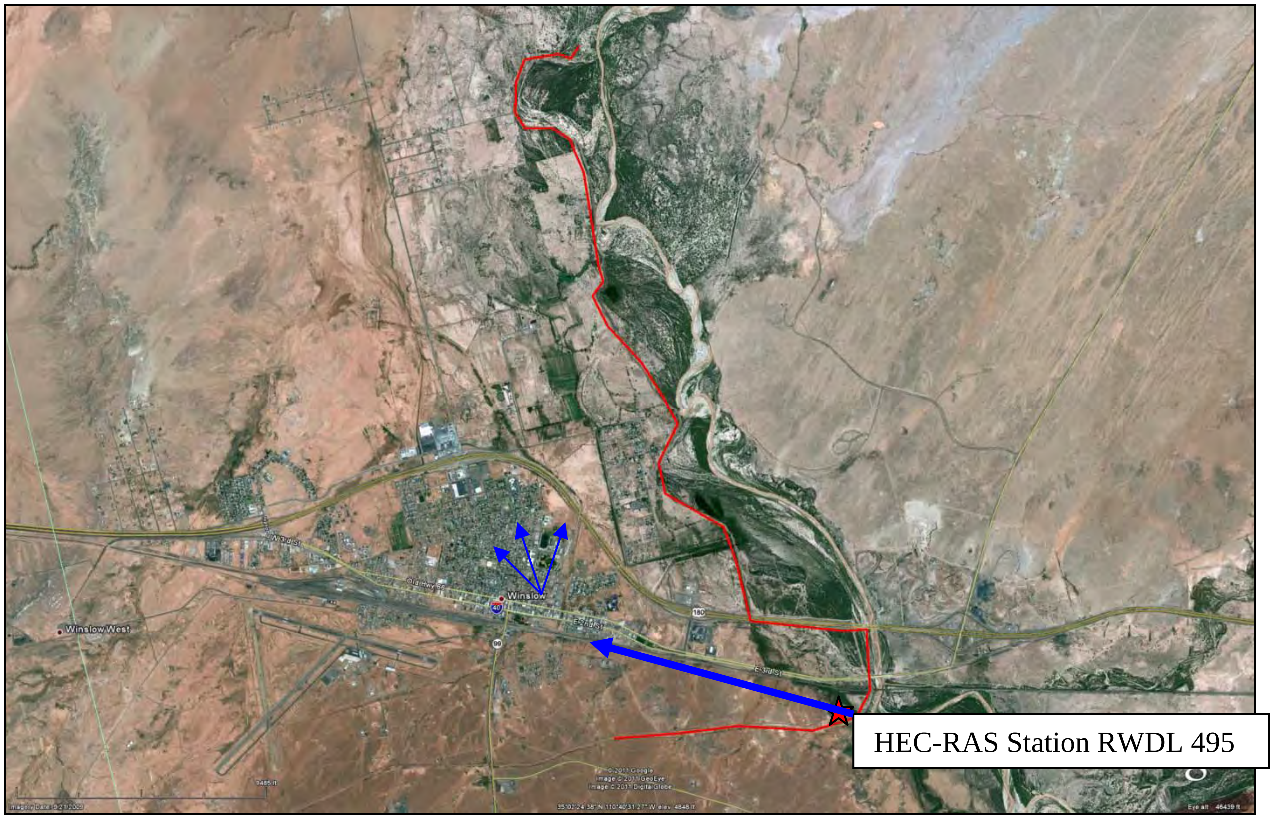


Figure 5. RWDL 495 Flood Direction

Table 1. RWDL 495 Hydraulic Loading

	Ruby Wash Diversion Levee (Crest Elevation 4867 ft)		
	HEC RAS 495.6543		
Year-Event	Q (cfs)	V (ft/s)	Water Surface Elevation (ft)
25.00	1580.00	1.10	4864.60
50.00	2330.00	0.22	4866.95
100.00	2860.00	0.17	4872.02
200.00	3110.00	0.17	4873.44

6.1.2 Past Performance

There is no available historical evidence that the levee has been breached at this location before. Visual observations in the field indicate highly erodible levee soils, as illustrated by rills and ruts in the levee slope. The RWDL at Station 495 is largely unchanged from its original construction, with an 18-foot wide crest, and 10-foot wide road and 2:1 (H:V) slopes.

6.1.3 Engineering Properties for Evaluation

The approach for determining levee reliability is generally to describe the levee geometry and engineering properties in terms of distributions of the engineering property values that are important in the analysis. Because of uncertainty in the foundation conditions near RWDL 495, two potential cross sections were developed, one with the levee supported on a bedrock foundation, and one supported on an alluvial soil foundation. The sections used for analysis are shown in Figures 6 and 7. RWDL 495 has a crest elevation of about 4867 feet, a crest width of about 18 feet and 2:1 (H:V) landside and riverside slopes. The height of the levee is generally about 8 to 10 feet above the adjacent grades.

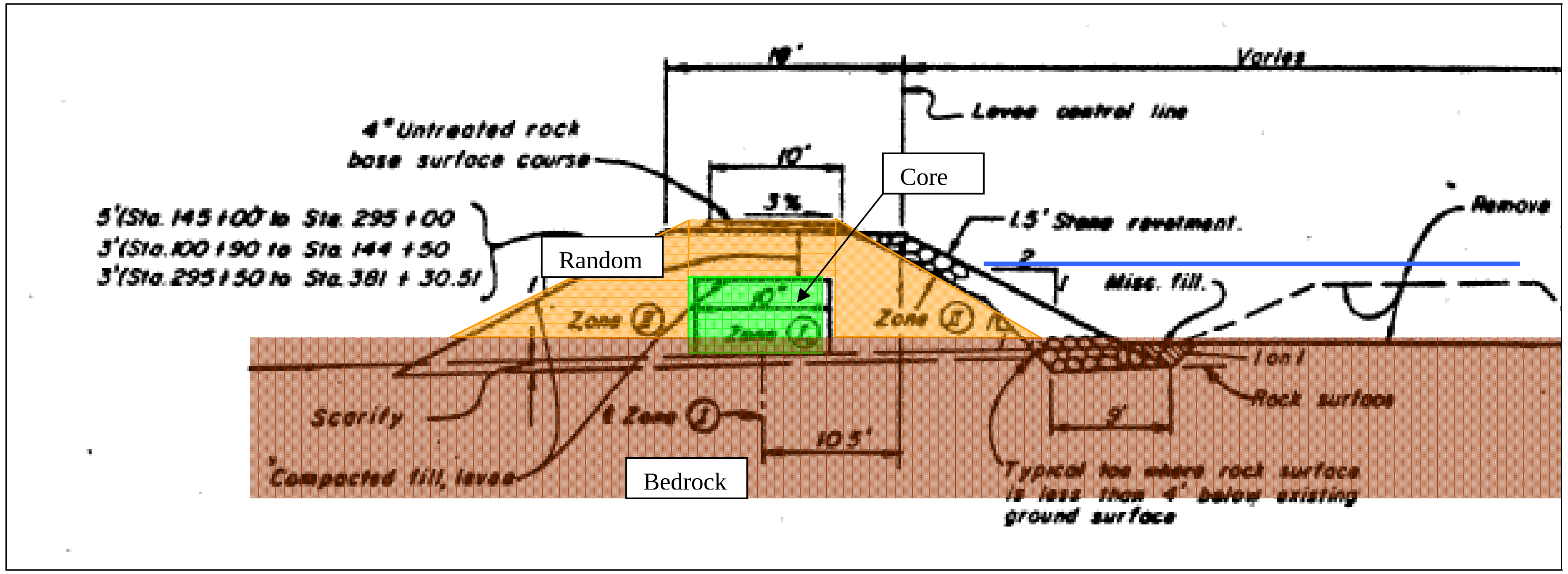


Figure 6. RWDL 495 Supported on Bedrock Foundation (RWDL Asbuilts, 1971)

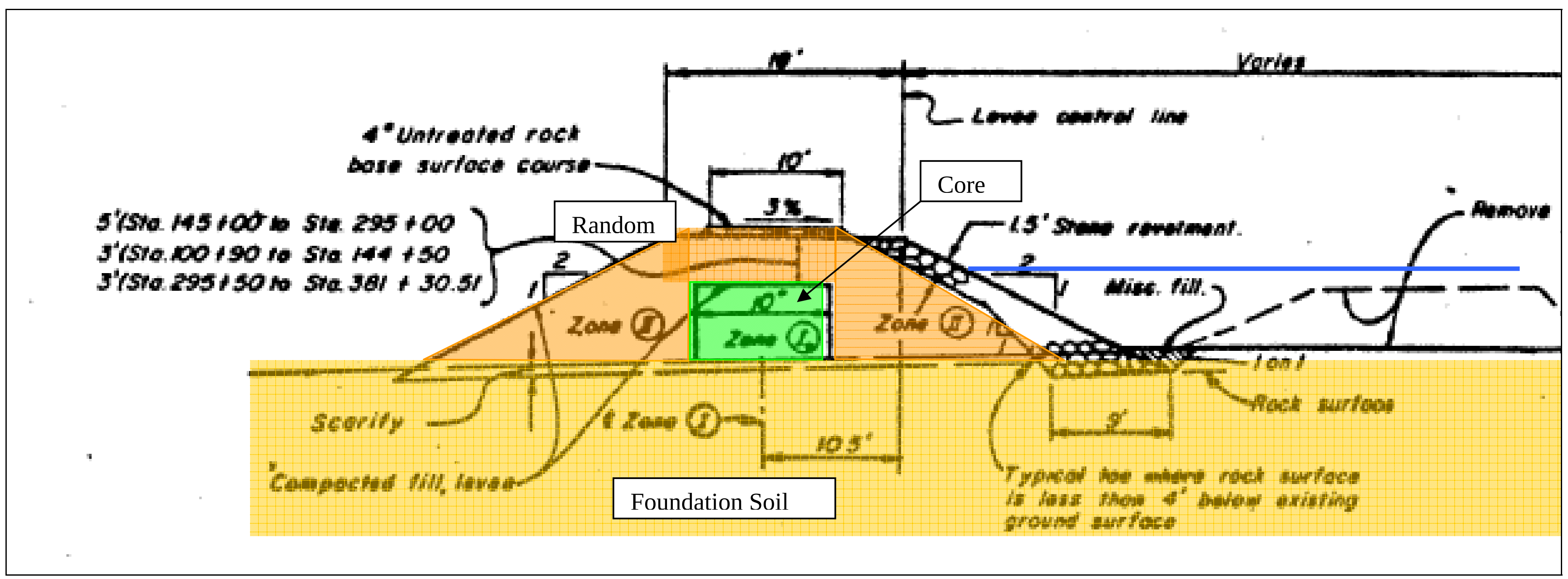


Figure 7. RWDL 495 Supported on Soil Foundation (RWDL Asbuilts, 1971)

Table 2 summarizes the engineering properties used for evaluation of seepage, slope stability, landside and riverside erosion failure modes. Engineering test data in the area of RWDL 495 was sparse; therefore a wide uncertainty was used to incorporate a wider possible range of performance. The engineering properties selected are supported by literature and consistent with explorations of the levee outside of the RWDL 495 area. The zoning and configuration of the levee are based on as-built record drawings and review of the construction contract documents.

Table 2. Engineering Property Distributions for Stability and Seepage Reliability Analysis Ruby Wash Diversion Levee RWDL 495

Soil Layer	Unit Weight (pcf)	Vertical Hydraulic Conductivity[2] (cm/sec)	Horizontal/Vertical Conductivity Ratio	Effective Friction Angle (degrees)	Layer Thickness
Zone 1 (Core)	120 120+σ=130 120-σ=110	$1x10^{-4}$ $1x10^{-4}+\sigma=1x10^{-3}$ $1x10^{-4}-\sigma=1x10^{-5}$	10 10+σ=25 10-σ=4	32 32+σ=35 32-σ=29	-
Zone 2 (Shell)	125 125+σ=135 125-σ=115	$1x10^{-2}$ $1x10^{-2}+\sigma=1x10^{-1}$ $1x10^{-2}-\sigma=1x10^{-3}$	4 4+σ=10 4-σ=1	36 36+σ=40 36-σ=32	-
Moenkopi Bedrock	130	$1x10^{-7}$	25 25+σ=100 25-σ=10	40 40+σ=42 40-σ=38	-
Foundation Blanket (clayey)	105 105-σ=90 105+σ=120	$5x10^{-4}$ $5x10^{-4}+\sigma=5x10^{-3}$ $5x10^{-4}-\sigma=5x10^{-6}$	10 10+σ=25 10-σ=4	32 32+σ=34 32-σ=30	7 7+σ=9 7-σ=5
Foundation Sand	102	$1x10^{-2}$	10	33	-

[2] Permeability values are not normally distributed, and are assumed to be more log-normal in nature.

Table 3. Property Distribution for Erosion Analysis

Soil Layer Description	Erodibility Coefficient (ft^3/lb-hr)	Critical Shear Stress
Levee Fill	1.87, coefficient of variation = 0.47	-
Foundation	1.87, coefficient of variation = 0.47	-
Stone Armor[3]	-	~ 6 psf

The soil layers shown in Tables 2 and 3 are described below:

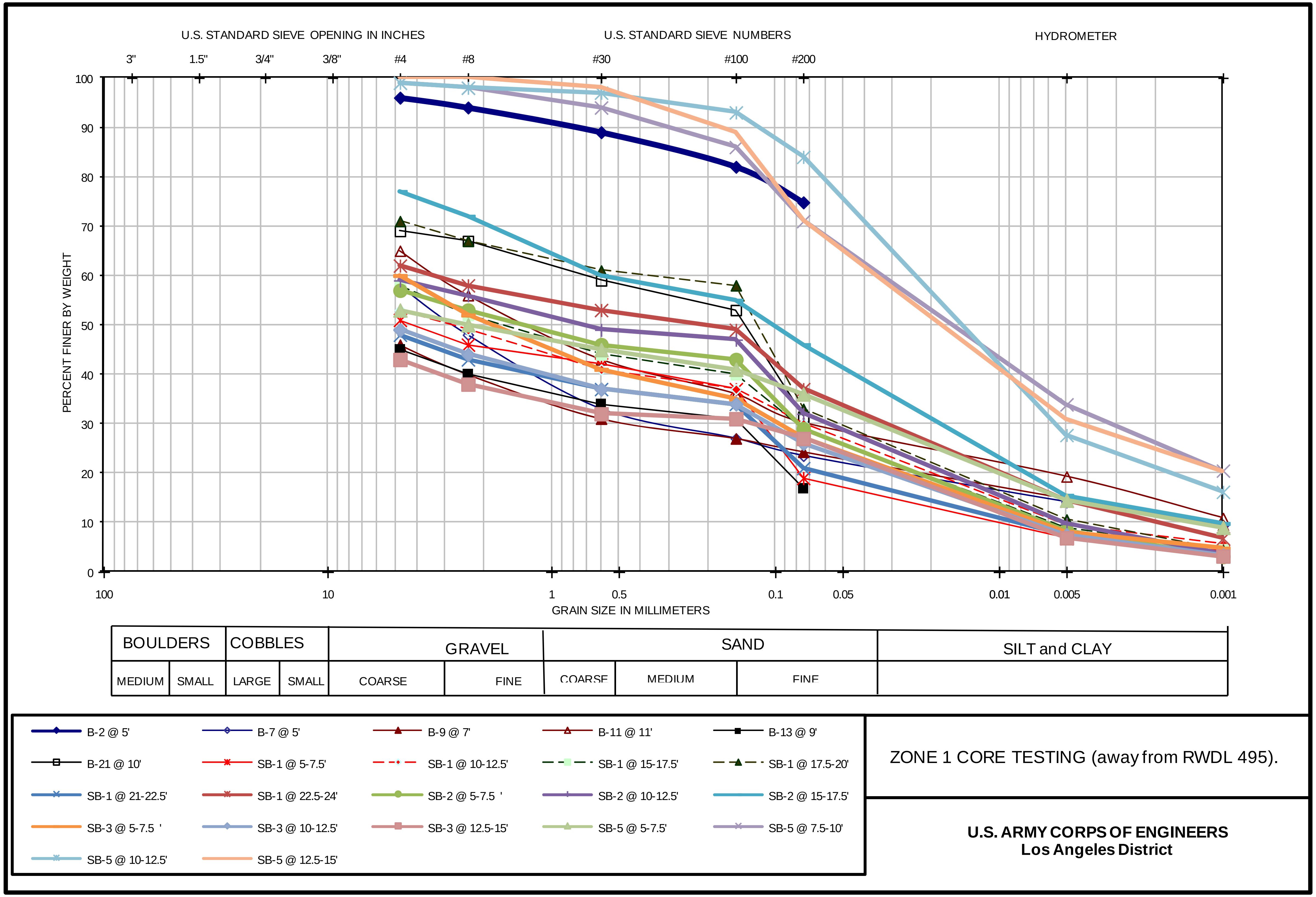
Zone 1 (Core): Zone 1 core material contract specifications were any on-site soils with at least 70 percent passing the No. 4 sieve. In general the soils are classified as lean clay to clayey and silty sands and gravels. Lab testing on the in-situ core materials near RWDL 495 was not available, however lab testing along other portions of the Ruby Wash Diversion Levee generally indicate that the specifications were followed. Table 4 summarizes the gradation test results for portions of the Ruby Wash Diversion Levee that have been tested; no applicable during-construction data is available to supplement the given soil data. The results indicate a wide range of classification, consistent with the estimated distribution of engineering parameters chosen

Zone 2 (Shell/Random Fill): Zone 2 shell material is generally described as gravel of various sizes with various amounts of finer grain soil. In general, Zone 2 was constructed of excavated Moenkopi foundation rock that was excavated and replaced as fill.

[3] Even a degraded armor stone (due to dissolution in water) is still expected to have critical shear velocities greater than the anticipated hydraulic loading.

Table 4. Zone 1 Core Testing (away from RWDL 495)

					Sieve Sizes: Percent Passing by Weight								
					GRAVEL	SAND				FINES	SILT	CLAY	
					Coarse & Fine	Coarse		Medium	Fine				
Boring Num.	Sample Depth Interval (feet)	USCS	LL	PI	1/4"	#4	#8	#30	#100	#200	0.005 mm	0.001 mm	Nat. Dry Den. (pcf)
B-2	5	ML	29	12	97	96	94	89	82	74.8	–	–	112.4
B-7	5	GM	23	8	62	58	48	33	27	23.5	14.1	9.4	–
B-9	7	GM	24	7	49	46	40	31	27	24.3	14.8	9.5	113.6
B-11	11	GM	23	6	70	65	56	43	36	30.2	19.3	10.9	113.4
B-13	9	GM	-	NP	47	45	40	34	31	16.9	–	–	–
B-21	10	SM	18	NP	71	69	67	59	53	31.4	–	–	118.6
SB-1	5-7.5	GM	-	NP	53	51	46	42	37	19	6.8	4.2	
SB-1	10-12.5	GC-GM	20	4	56	53	49	41	37	30	9.5	5.7	
SB-1	15-17.5	GC-GM	19	5	61	58	52	44	40	29	8.9	4.6	
SB-1	17.5-20	SM	17	3	73	71	67	61	58	33	10.6	4.4	
SB-1	21-22.5	GC-GM	16	7	51	48	43	37	34	21	7.5	3.2	
SB-1	22.5-24	GM	18	3	65	62	58	53	49	37	14.4	6.8	
SB-2	5-7.5	GM	18	2	59	57	53	46	43	29	8.4	3.1	
SB-2	10-12.5	GM	19	3	63	59	56	49	47	32	9.7	3.6	
SB-2	15-17.5	SC-SM	21	5	78	77	72	60	55	46	15.5	9.8	
SB-3	5-7.5	GC	19	3	63	60	52	41	35	27	8.2	4.8	
SB-3	10-12.5	GC-GM	20	4	52	49	44	37	34	26	7.5	3.4	
SB-3	12.5-15	GC-GM	22	4	46	43	38	32	31	27	6.9	3.2	
SB-5	5-7.5	GC	25	9	56	53	50	45	41	36	14.5	9.0	
SB-5	7.5-10	CL	26	10	99	99	98	94	86	71	33.9	20.5	
SB-5	10-12.5	CL	25	8	99	99	98	97	93	84	27.7	16.2	
SB-5	12.5-15	CL	24	9	100	100	100	98	89	71	30.8	20.3	



U.S. STANDARD SIEVE OPENING IN INCHES
U.S. STANDARD SIEVE NUMBERS
HYDROMETER
3"
1.5"
3/4"
3/8"
#4
#8
#30
#100
#200
PERCENT FINER BY WEIGHT
100
90
80
70
60
50
40
30
20
10
0
100
10
1
0.5
0.1
0.05
0.01
0.005
0.001
GRAIN SIZE IN MILLIMETERS
BOULDERS
COBBLES
GRAVEL
SAND
SILT and CLAY
MEDIUM
SMALL
LARGE
SMALL
COARSE
FINE
COARSE
MEDIUM
FINE
B-2 @ 5'
B-7 @ 5'
B-9 @ 7'
B-11 @ 11'
B-13 @ 9'
B-21 @ 10'
SB-1 @ 5-7.5'
SB-1 @ 10-12.5'
SB-1 @ 15-17.5'
SB-1 @ 17.5-20'
SB-1 @ 21-22.5'
SB-1 @ 22.5-24'
SB-2 @ 5-7.5 '
SB-2 @ 10-12.5'
SB-2 @ 15-17.5'
SB-3 @ 5-7.5 '
SB-3 @ 10-12.5'
SB-3 @ 12.5-15'
SB-5 @ 5-7.5'
SB-5 @ 7.5-10'
SB-5 @ 10-12.5'
SB-5 @ 12.5-15'
ZONE 1 CORE TESTING (away from RWDL 495).
U.S. ARMY CORPS OF ENGINEERS
Los Angeles District

Moenkopi Bedrock: Moenkopi is described in the geology section of this report. In the area of the Ruby Wash Diversion Levee the Moenkopi bedrock was observed in outcrops to primarily consist of sandstone that has some cementation that may be soluble in water. Rock joints were mostly horizontal, consistent with the sedimentary nature of the deposit. The Moenkopi is assumed to have low permeability in the vertical direction. Moenkopi Formation bedrock is shallow in the foundation throughout most of RWDL, often just a few inches to 2 ft deep below sand residuum. But at the LCR junction, LCR channel erosion and meandering over time has more deeply eroded the Moenkopi at the downstream RWDL foundation. Drill data to prove depth is sparse, but the depth to bedrock in the downstream areas of RWDL, and in particular at RWDL 495, is expected to be several feet to several tens of feet below the ground surface.

Clayey Foundation Blanket: Stick logs of borings performed near the railroad bridge just downstream of RWDL 495 indicate that at some locations the alluvium consists of a clayey layer over more sandy layers. The logs indicate the clayey layer is about 7 feet thick, however variations are anticipated. The clayey alluvium has an unknown stress history and gradation. The wide band in the hydraulic conductivities was selected to model the estimated range of possible engineering properties that are likely to be encountered in foundation soils near RWDL 495.

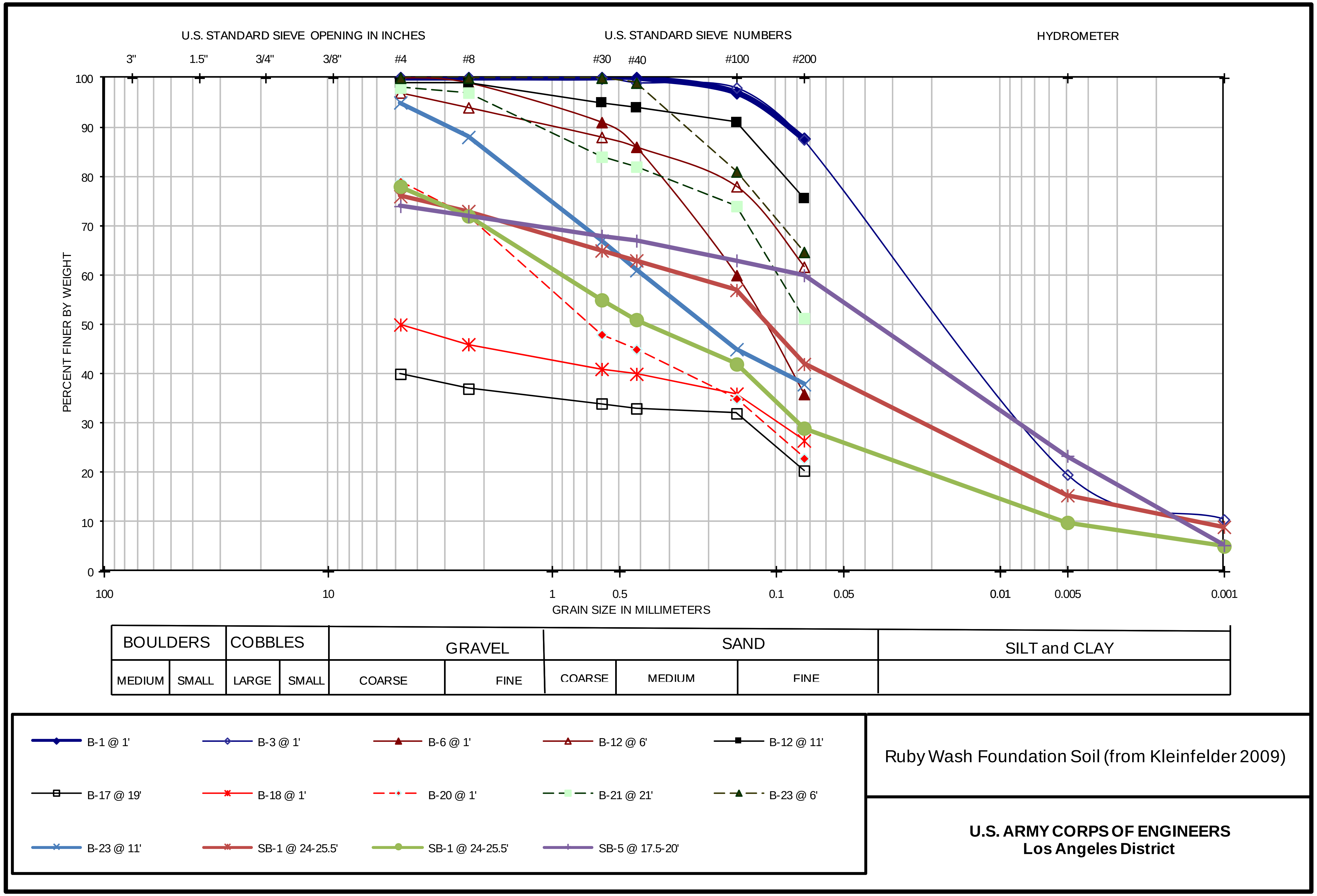
Foundation Sands: Sands were shown in stick logs below the surficial clays. In general, sand gradation throughout the project range from very fine to coarse. For the underseepage evaluation, the range in possible sand permeabilities is relatively unimportant as long as the permeability is much higher than the overlying clay permeability.

Table 5 Summarizes gradation data from foundation soils at other Ruby Wash locations. In some locations the foundations soils may be weathered Moenkopi bedrock. In general, a wide range of permeability and engineering properties should be anticipated.

Table 5. Ruby Wash Foundation Soil Classification.[4]

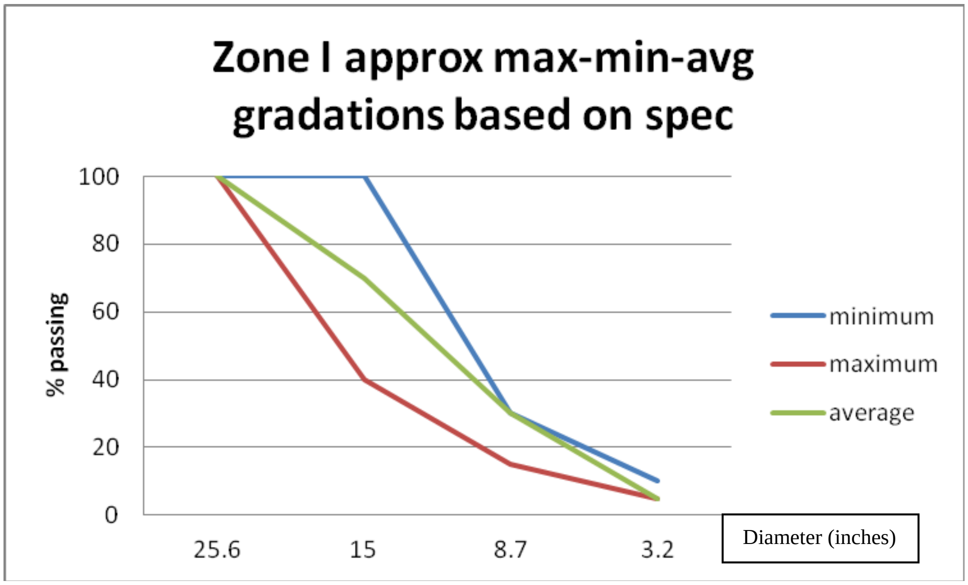
					Sieve Sizes: Percent Passing by Weight										
					GRAVEL Coarse & Fine	SAND Coarse		SAND Medium		SAND Fine		FINES	SILT	CLAY	
Boring Num.	Sample Depth (feet)	USCS	LL	PI	1/4"	#4	#8	#10	#30	#40	#100	#200	0.005 mm	0.001 mm	Nat. Dry Den. (pcf)
B-1	1	ML	29	13	100	100	100	100	100	100	97	87.7	–	–	–
B-3	1	ML	27	11	100	100	100	100	100	99	98	87.5	19.6	10.5	100.3
B-6	1	SM	20	1	100	100	99	99	91	86	60	35.9	–	–	101.3
B-12	6	ML	21	6	98	97	94	94	88	86	78	61.7	–	–	–
B-12	11	ML	25	6	100	99	99	98	95	94	91	75.6	–	–	–
B-17	19	GM	–	NP	42	40	37	36	34	33	32	20.4	–	–	124.8
B-18	1	GM	22	7	52	50	46	45	41	40	36	26.5	–	–	–
B-20	1	SM	17	1	81	79	72	70	48	45	35	22.9	–	–	104.9
B-21	21	ML	21	6	99	98	97	96	84	82	74	51.3	–	–	–
B-23	6	ML	25	5	100	100	100	100	100	99	81	64.7	–	–	–
***B-23**	**11**	**SM**	**–**	**NP**	**96**	**95**	**88**	**86**	**67**	**61**	**45**	**37.9**	**–**	**–**	**85.4**
SB-1	24-25.5	SM	17	2	77	76	73	72	65	63	57	42	15.4	9.0	
SB-1	25.5-26	SM	18	NP	80	78	72	71	55	51	42	29	9.9	5.1	
SB-5	17.5-20	CL	31	12	74	74	72	72	68	67	63	60	23.4	5.3	

[4] Kleinfelder 2009



Graph 2. Ruby Wash Foundation Soil Gradations (from Kleinfelder 2009)

Rip Rap Armor: The Ruby Wash Diversion Levee has armor located on the riverside of the levee at RWDL 495. The gradations indicate a mean stone size of about 12 inches. Graph 1 shows the range of armor gradations for the Ruby Wash portions of the project near RWDL 495. The rip rap is primarily broken up rock of the Moenkopi formation. The formation is highly subject to weathering including dissolution of cementing bonds during wetting cycles. The geotechnical team thought that reduction of stone weight by about 30 percent over 50 years could be possible due to the extreme weathering.



Graph 3. Moenkopi Rip Rap Gradation near RWDL 495

Using the average gradation from Graph 3 and Equation 1 (below), the critical velocity for 12-inch diameter Moenkopi is about 8.9 feet per second. Stream velocities are estimated to be much less than this critical velocity at this location.

(Equation 1) $$V = C[2g\frac{\gamma_s - \gamma_w}{\gamma_w}]^{1/2}(D_{50})^{1/2}$$ (From USACE EM 1110-2-1601)

V= critical velocity (ft/sec)
C= Isbash constant (0.85 to 1.2 for high to low turbulence)
g=acceleration of gravity (ft/sec^2)
D_{50}= diameter of rock with 50 percent smaller (ft)
γ_s=unit weight of stones (pcf)
γ_w=unit weight of water (pcf)

Example calculation:
D_{50}=12 inches=1 ft, γ_s=140 pcf, γ_w=62.4 pcf, C=1.0; therefore
V= 8.9 ft/sec

6.2 HEC-RAS Winslow Station 51500

HEC-RAS Winslow Station 51500 is located near the railroad and highway crossings. This point was chosen as an index point because the levee is relatively narrow, has a relatively lower crest elevation, does not have landside slope armoring and does not have complete riverside armoring. The levee soils observed in the field were fine sand and considered to be highly erodible. Winslow Station 51500 has a crest width of about 18 feet, a crest elevation of about 4865 feet and 2:1 (H:V) landside and riverside slopes. The levee height is generally about 10 feet. The geometry is based on current topographic survey. The levee has been raised and reconstructed from original construction as outlined in the 1980s ADOT plans using random fill. It is speculated that the original levee may have been constructed over some looser random fill, and that these layers may remain. The levee includes a seepage cutoff at this locations.

6.2.1 Hydraulic Loading and Consequences

The geotechnical team understands that the flooding consequences that would result from a breach at Winslow Station 51500 are similar to the flood map that would result from a breach at RWDL 495. Because of this, the "weak link" should be chosen to determine flood frequency for cost/benefit analysis. Figure 8 shows the approximate direction of flooding as a result of a levee breach at Winslow Station 37000. Table 6 shows the estimated hydraulic loading for those index points evaluated along the Winslow Levee. The Winslow Station 51500 loading is shown in the first column. Because the levee is set back from the main channel flow, average channel flows were used in the erosion evaluation at Winslow Station 51500.

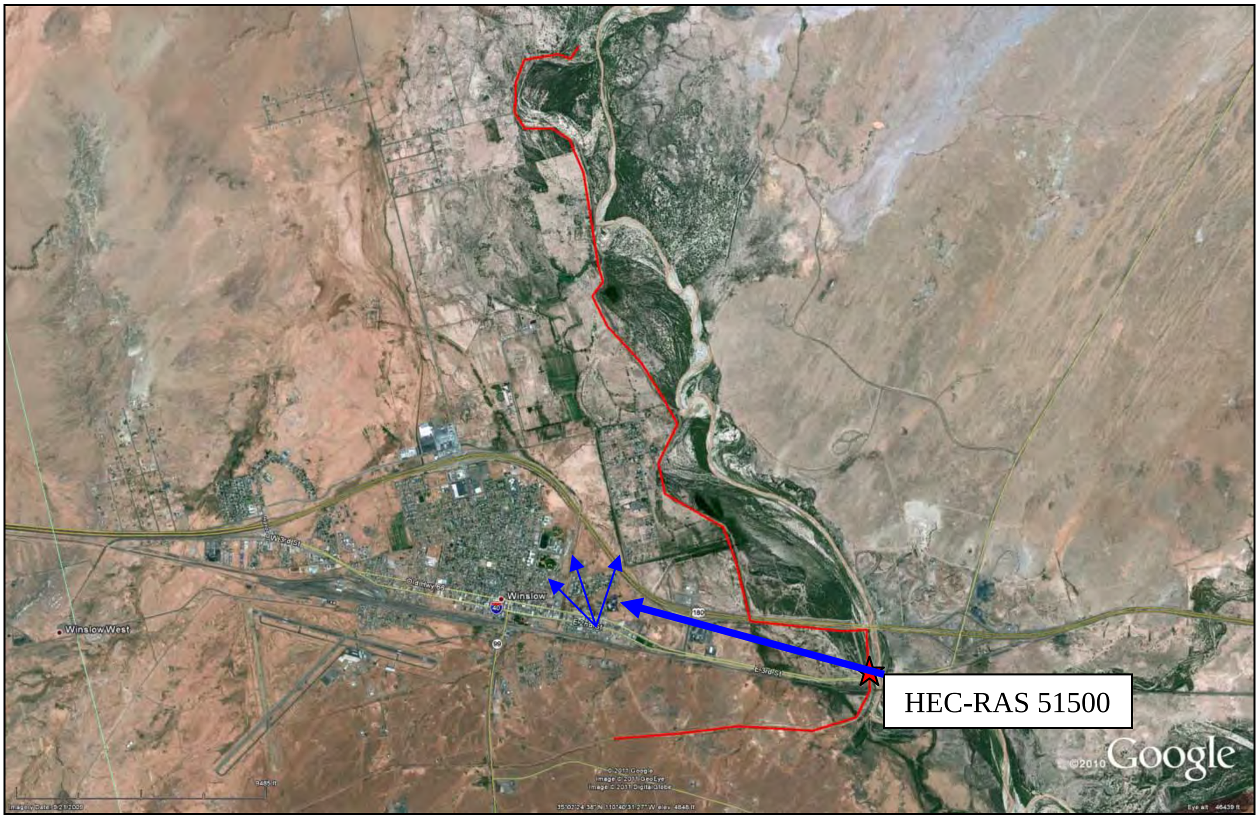


Figure 8. Winslow Station 51500 Flood Direction

Table 6. Hydraulic Loading for Winslow Levee Sections

		Winslow Levee HEC RAS								
		51500 Crest Elevation 4865 feet			37000 Crest Elevation 4855 feet			29000 Crest Elevation 4849 feet		
Year-Event	Q (cfs)	V (ft/s) in total cross-section	V (ft/s) in main channel	Water Surface Elevation (ft)	V (ft/s) in total cross-section	V (ft/s) in main channel	Water Surface Elevation (ft)	V (ft/s) in total cross-section	V (ft/s) in main channel	Water Surface Elevation (ft)
2	8070.00	2.9700	4.6200	4854.42	2.27	2.41	4845.93	1.88	5.42	4840.61
5	16360.00	3.19	6.14	4856.73	2.56	3.00	4847.28	1.52	6.12	4842.22
10	24400.00	3.32	7.21	4858.27	2.86	3.43	4848.27	1.52	6.34	4843.28
25	38310.00	3.58	8.21	4860.43	3.31	4.03	4849.68	1.63	6.64	4844.75
50	52020.00	3.85	8.93	4862.25	3.67	4.49	4850.87	1.75	6.9	4845.96
100	69200.00	4.22	9.80	4864.12	4	4.97	4852.07	2.46	9.77	4845.65
200	90660.00	4.44	10.29	4866.69	4.44	5.52	4853.46	2.72	10.53	4846.67
500	127250.00	1.19	2.49	4871.29	5.56	6.94	4854.52	3.1	11.59	4848.18

6.2.2 Past Performance

It is not believed that the levee has been breached at this location before. Visual observations in the field indicate highly erodible levee soils, as illustrated by rills and ruts in the levee slope.

6.2.3 Engineering Properties for Evaluation

Winslow Station 51500 has four relevant conditions: a bentonite seepage cutoff, a sand levee fill, a clay blanket, and foundation sands. These layers are taken from geotechnical exploration stick logs from a 1990s Dames and Moore exploration and shown on the as-built plans. Engineering lab tests were not available at Winslow Station 51500, so literature was used to support the parameters shown in Table 7. Graphs 2 and 3 illustrate the basis for engineering parameter selection for the Winslow levees. For the seepage analysis, the absolute values of the hydraulic conductivity are less important than the contrast in permeability between soil layers. Where standard deviations are not shown, the analysis was determined to be insensitive to those parameters.

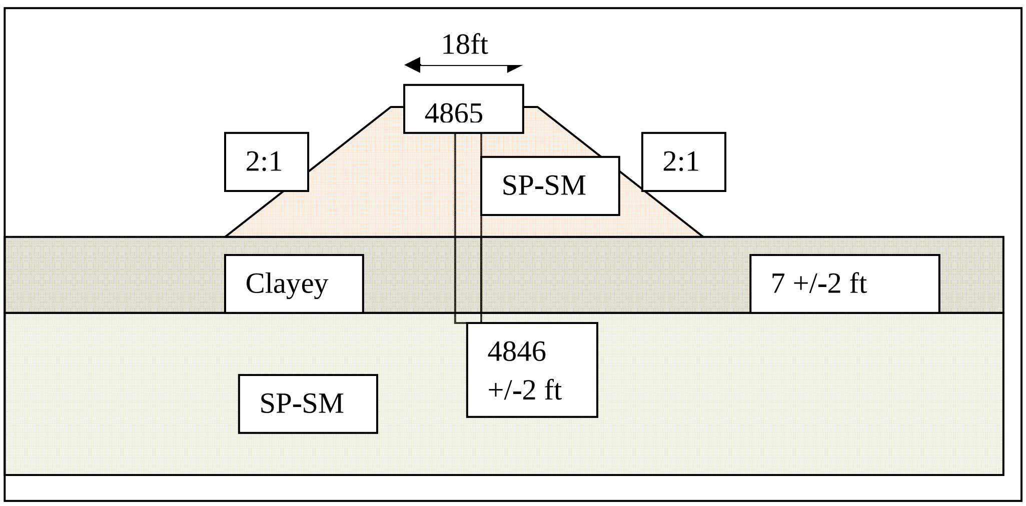


Figure 9. Schematic Cross Section – Winslow Station 51500

Levee Fill (SP-SM): Levee fill was constructed of previous levee fills and material excavated from the nearby channel. Because the near surface soils predominantly consist of sand and silt and the observed levee fill was sand and silt at the ground surface, the engineering properties were assumed to be similar to that of an SP or SM classification. The levee was assumed to have somewhat higher strength than the natural deposits because of compaction effort; therefore a moderately higher friction angle was considered warranted. The levee fill is considered to be highly erodible.

Sand Foundation (SP-SM): Near surface foundation soils predominantly consist of sand and silt, and the observed levee fill was also sand and silt at the ground surface. Because of the channel nature, it is assumed the soils are in a loose and relatively unconsolidated state. Foundation soils are highly erodible. Some rip rap was observed along the riverside levee toe; however it did not appear of high quality or continuous, and was not considered to provide significant erosion protection benefits in the reliability analysis.

Foundation Blanket (Clayey): Stick logs of the foundation indicate that the levee is likely supported on a relatively less permeable layer with descriptions ranging from SM to CL-SM. This indicates that the foundation layer is likely significantly less permeable than the underlying sands, especially where classified as CL.

Bentonite Cutoff: As-built drawings indicate that a bentonite slurry was used to construct a seepage cutoff down the center of the levee. The analysis was most sensitive to the tip elevation of the cutoff, and was assumed to have a standard deviation of 2 feet around the expected values.

Table 7. Engineering Properties Used for Stability and Seepage Analysis – Winslow Station 51500

Soil Layer	Unit Weight (pcf)	Vertical Hydraulic Conductivity[5] (cm/sec)	Horizontal/ Vertical Conductivity Ratio	Effective Friction Angle (degrees)	Layer Thickness (ft)
Bentonite cutoff	95	Less than $1x10^{-5}$	1	22 c= 50 psf	Bottom elev = 4846 ft, σ=2ft
Sand Levee Fill (SP-SM)	125 125+σ=135 125-σ=115	$1x10^{-2}$ $1x10^{-2}$+σ=$1x10^{-1}$ $1x10^{-2}$-σ=$1x10^{-3}$	5 5+σ=25 5-σ=2	36 36+σ=40 36-σ=32	-
Foundation Blanket (clayey)	105 105-σ=90 105+σ=120	$5x10^{-4}$ $5x10^{-4}$+σ=$5x10^{-3}$ $5x10^{-4}$-σ=$5x10^{-6}$	10 10+σ=25 10-σ=4	32 32+σ=34 32-σ=30	7 7+σ=9 7-σ=5
Foundation Sand (SP-SM)	102 102-σ=95 102+σ=110	$1x10^{-2}$ $1x10^{-2}$+σ=$1x10^{-1}$ $1x10^{-2}$-σ=$1x10^{-3}$	10	33 33+σ=36 33-σ=30	-

Table 8. Property Distribution for Erosion Analysis

Soil Layer Description	Erodibility Coefficient (ft^3/lb-hr)	Armor Critical Shear Stress
Levee Fill	1.87, coefficient of variation = 0.47	-
Foundation	1.87, coefficient of variation = 0.47	-

[5] Permeability values are not normally distributed, and are assumed to be more log-normal in nature.

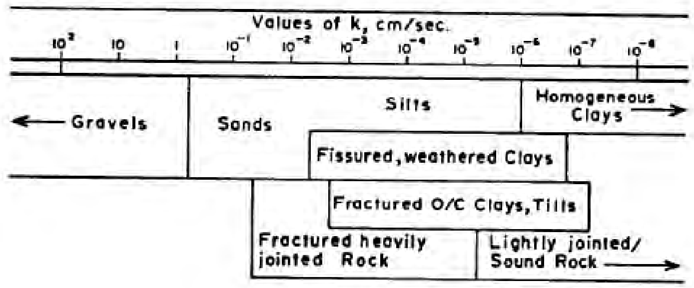
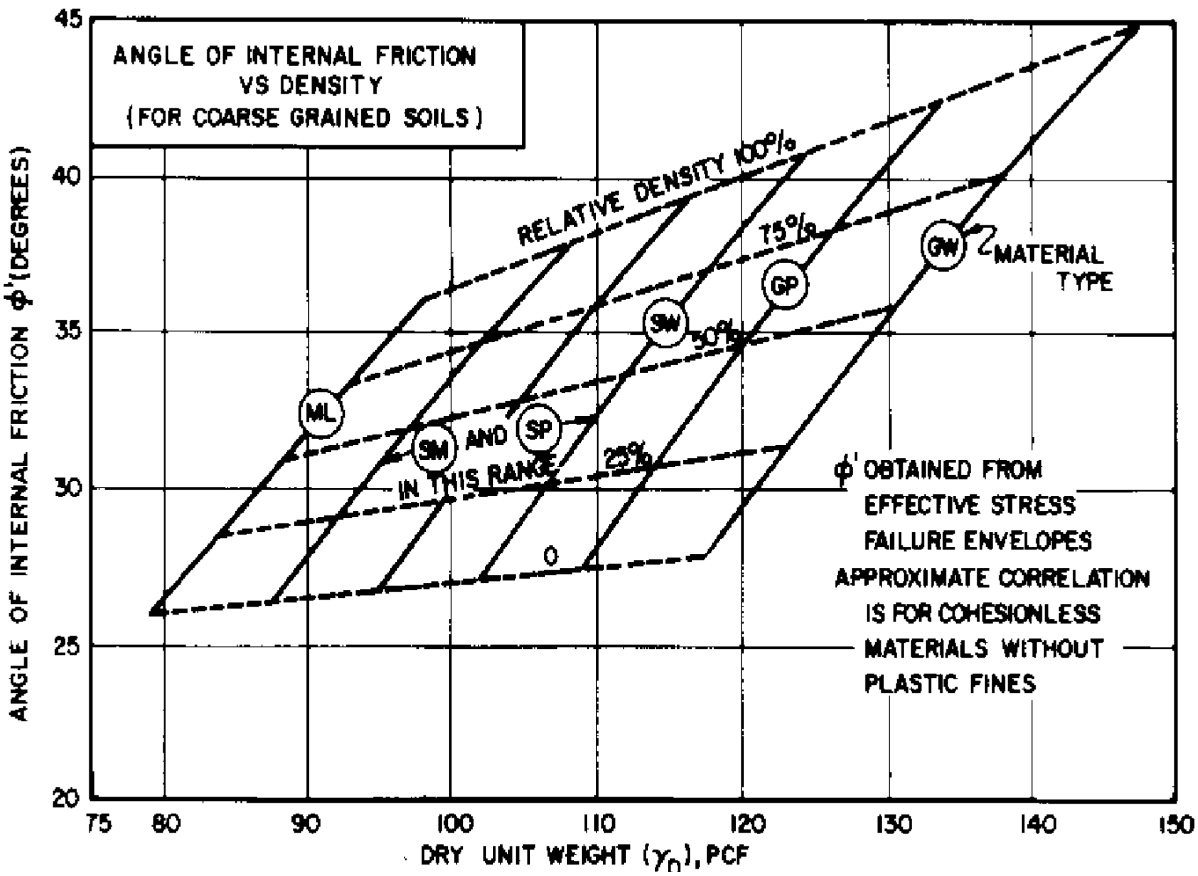


Figure 2-5. Approximate range in coefficient of permeability of soils and rocks (from Milligan[224])

Graph 4. Basis for Permeability Value Selection Winslow Levee Sections



Graph 5. Basis for Shear Strength and Unit Weight Selection Winslow Levees

6.3 HEC-RAS Winslow Station 37000

Winslow Station 37000 was selected as a potential failure location due to the strong tendency of the Little Colorado River to meander to the west at this location and create an impinging flow condition near the levee. It can be seen from aerial photography that the levee crosses old meanders in the area. Winslow Station 37000 has a crest elevation of approximately 4855 feet, a 22 foot crest width, approximately 2:1 (H:V) landside slopes, and 2:1 riverside slopes. The levee has been modified at this location to a higher elevation, rip-rap protection on both sides of the levee and has been widened to 22 feet. The geometry in the analysis is based on current topographic survey information. Major upgrades occurred to the levee in the 1980s as shown on the ADWR plans. The major reconstruction was completed in 1989 with the goals of raising levees, flattening slopes, construction impermeable core and armoring the levee.

6.3.1 Hydraulic Loading and Consequences

The geotechnical team understands that the flooding consequences that would result from a breach at Winslow Station 37000 are different than flooding as a result of breach at RWDL 495 or Winslow 51500. Figure 10 shows the approximate flooding direction as a result of a levee breach at Winslow Station 37000. Flooding from the index point at Winslow Station 29000 may overlap flooding from Winslow Station 37000. Economic incremental analysis may help avoid “double counting” damages. The estimated hydraulic loading for Winslow Station 37000 is shown in the second column of Table 6. The main active portion of the channel is located adjacent to the levee at this location. River currents used for riverside erosion modeling were assumed to be consistent with the “main channel” flow velocities.

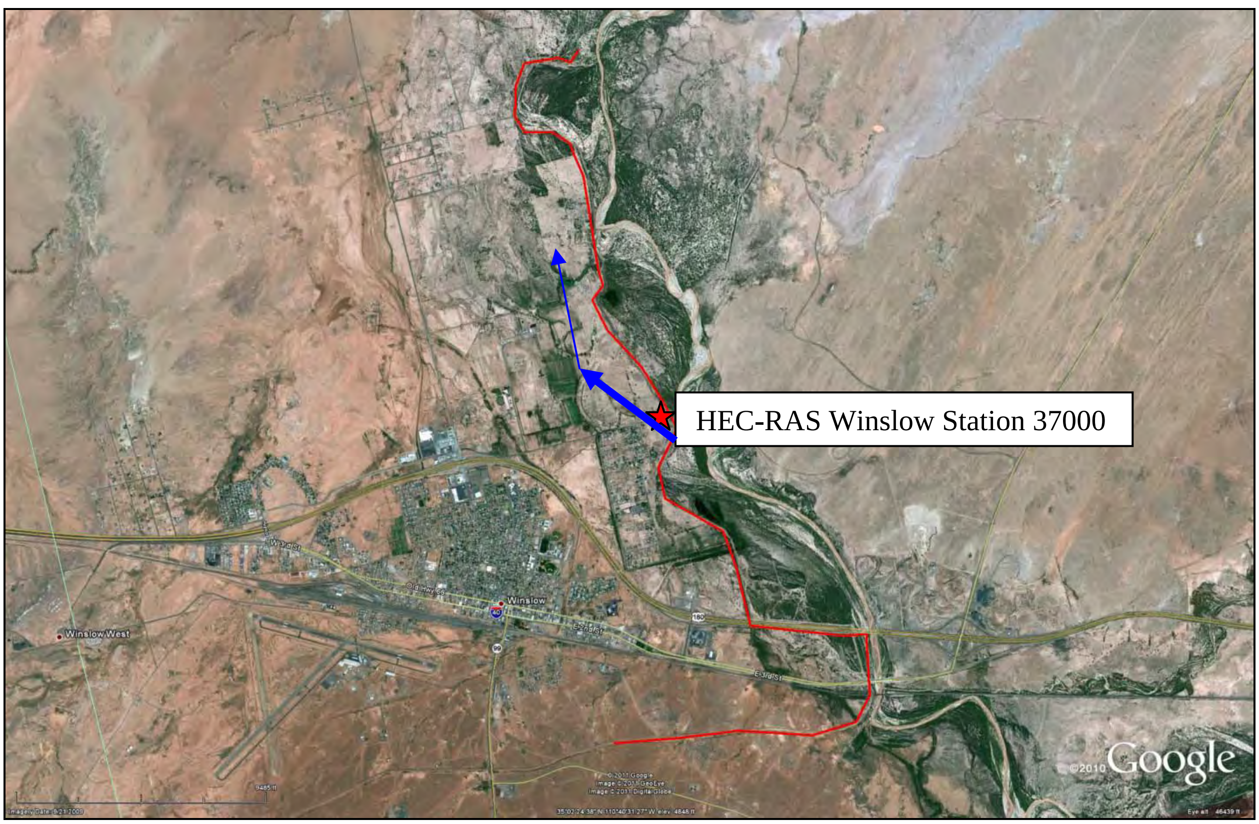


Figure 10. Direction of Flooding from Winslow Levee Station 37000

6.3.2 Past Performance

Due to the meandering of the river, erosion and levee damage as a result of river impingement have been of significant concern for many years at this location. Rip rap has been added on both sides of the levee, the crest heights have been increased, the crest widths have been increased and channel grading modifications have been performed to try to re-direct the Little Colorado River flow. As noted in the F3 geotechnical appendix report, this area has been previously overtopped, and during repairs was raised to its current elevation.

6.3.3 Engineering Properties for Evaluation

Winslow Station 37000 has four relevant layers for analysis, a bentonite seepage cutoff, a generally clayey levee fill, and foundation sands. Below the foundation sands a clayey layer is encountered. These layers were estimated from geotechnical exploration stick logs from a 1990s Dames and Moore exploration and shown on the as-built plans. Engineering lab tests were not available at Winslow Station 37000 so literature was used to support the parameters shown below. Figure 11 illustrates the general soil profile for Winslow Station 37000. Graphs 2 and 3 illustrate the basis for engineering parameter selection for the Winslow levees. For the seepage analysis, the absolute values of the hydraulic conductivity are less important than the contrast in permeability between soil layers. Where standard deviations are not shown, the analysis was determined to be insensitive to those parameters. Tables 9 and 10 summarize the material properties used in the engineering analysis. Graph 4 shows the rip-rap gradation tests for the basalt rip-rap along the Winslow Levee.

Levee Fill: Levee fill was constructed of previous levee fills and channel materials excavated from the nearby channel. The levee fill is generally thought to be more clayey in nature at this location. The levee was assumed to have somewhat higher strength than the natural deposits because of compaction effort; therefore moderately higher friction angle was warranted. The levee fill is considered moderately resistant to erosion. The levee is armored on both sides with approximately 12-inch basalt rip-rap that is anticipated to resist channel velocities in excess of 7 feet/second.

Sand Foundation: Near surface soils predominantly consist of sand and silt and the observed levee fill was sand and silt at the ground surface, soils. Because of the channel nature, it is assumed the soils are in a loose and relatively unconsolidated state. Foundation soils are considered highly erodible.

Bentonite Cutoff: As-built drawings indicate that a bentonite slurry was used to construct a seepage cutoff down the center of the levee. The analysis was most sensitive to the tip elevation of the cutoff which was assumed to have a standard deviation of 2 feet around the expected values.

Table 9. Engineering Properties Used for Stability and Seepage Analysis – Winslow Station 37000

Soil Layer	Unit Weight (pcf)	Vertical Hydraulic Conductivity (cm/sec)[6]	Horizontal/Vertical Conductivity Ratio	Effective Friction Angle (degrees)	Layer Thickness (ft)
Bentonite cutoff	95	Less than $1x10^{-5}$	1	22 c= 50 psf	Bottom elev = 4846 ft, σ=2ft
Clayey Levee Fill	115 115+σ=125 115-σ=105	$1x10^{-6}$ $1x10^{-6}$+σ=$1x10^{-5}$ $1x10^{-6}$-σ=$1x10^{-7}$	10 10+σ=25 10-σ=5	32 32+σ=35 32-σ=29	-
Foundation Sand (SP-SM)	102 102-σ=95 102+σ=110	$1x10^{-2}$ $1x10^{-2}$+σ=$1x10^{-1}$ $1x10^{-2}$-σ=$1x10^{-3}$	5 5+σ=10 10-σ=2	34 34+σ=37 34-σ=31	-

Table 10. Property Distribution for Erosion Analysis

Soil Layer Description	Erodibility Coefficient (ft^3/lb-hr)	Critical Stream Velocity
Levee Fill	0.094, coefficient of variation = 0.8	-
Foundation	1.87, coefficient of variation = 0.47	-
Basalt Rip Tap		7 ft/sec

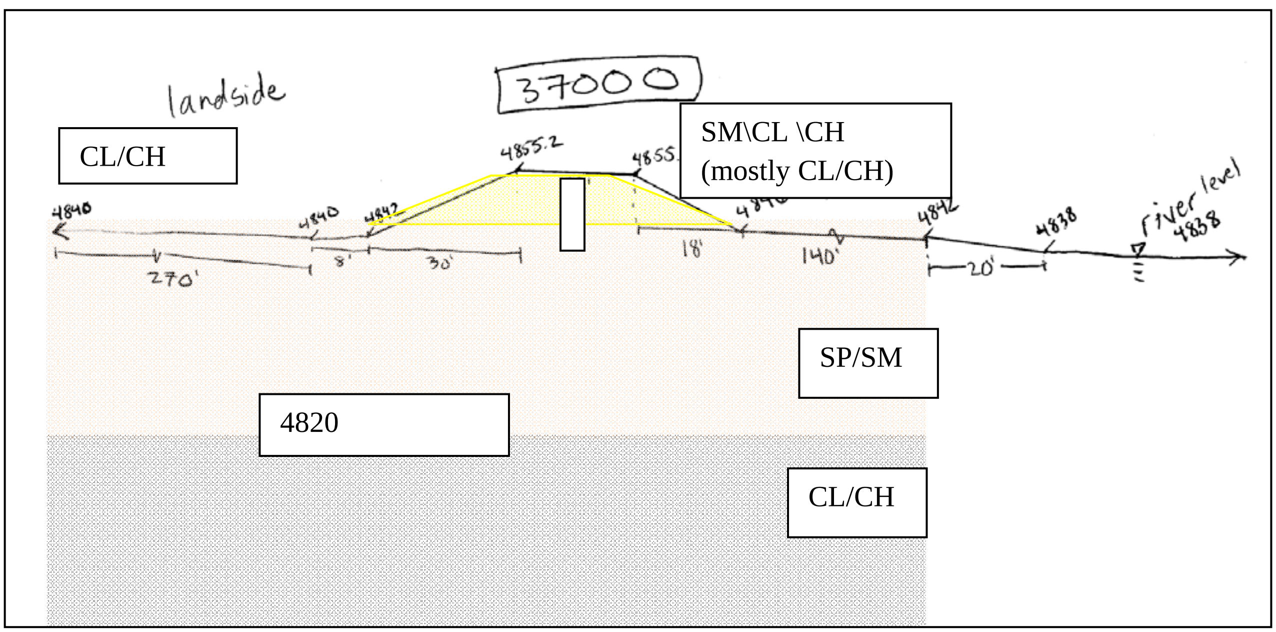
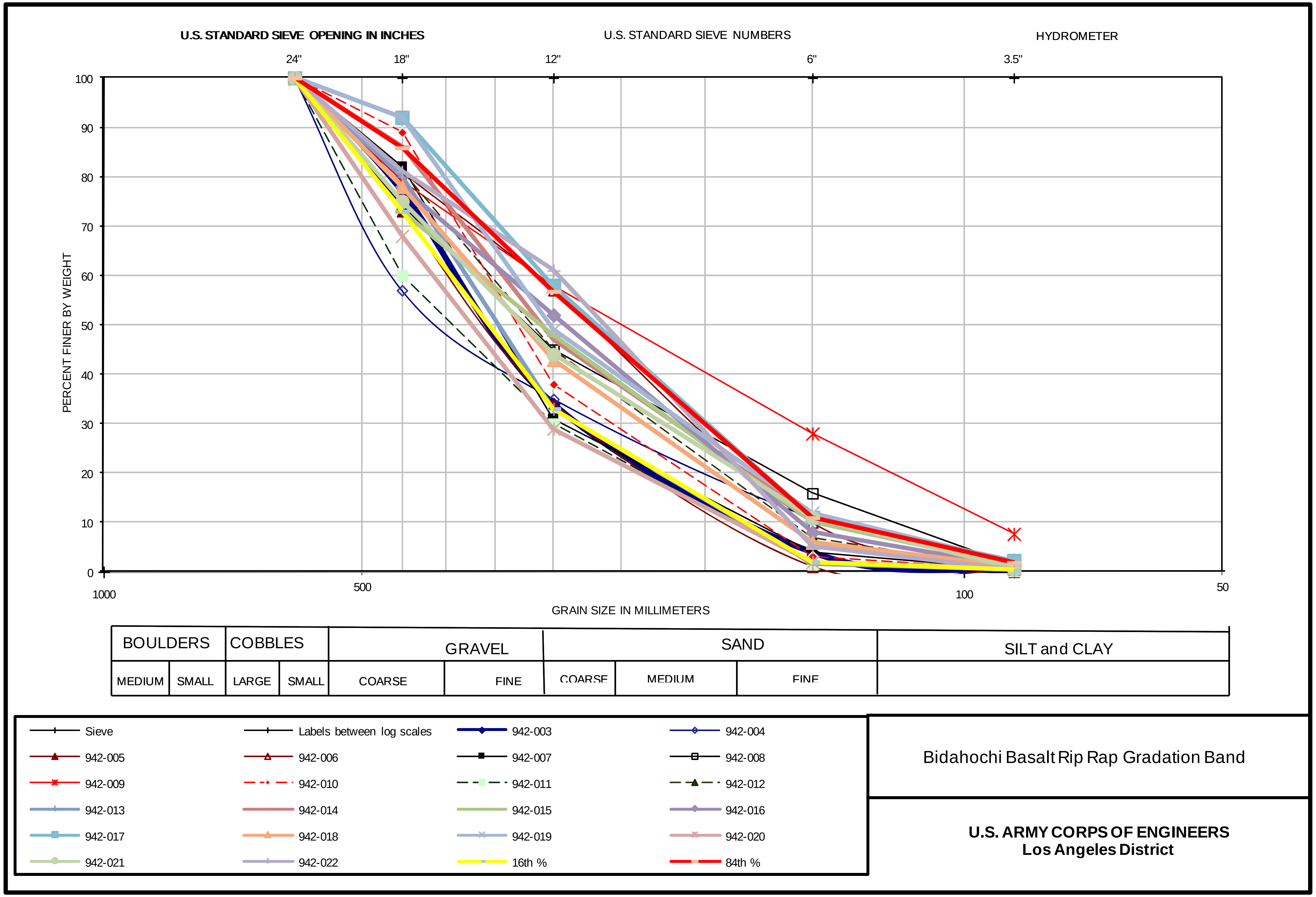


Figure 11. Levee Section Schematic at 37000

[6] Permeability values are not normally distributed, and are assumed to be more log-normal in nature.



Graph 6. Bidahochi Basalt Rip Rap Gradation Band

6.4 HEC-RAS Winslow Station 29000

Winslow Station 29000 was selected as a potential failure location, primarily because it has experienced unsatisfactory seepage performance before, as evidenced by observed sand boils and piping in multiple events, but most recently in 2004. Winslow Station 29000 had a crest width of about 18 feet, a crest elevation of about 4849 feet, and 2:1(H:V) landside and riverside slopes. This section of the levee has been modified in the late 1980s and then again after an overtopping breach occurred in 1993 and re-repaired after 2004 piping was observed. The conditions modeled assume that that foundation conditions that led to the 2004 piping incident may occur at other nearby adjacent areas. The upgrades have included construction taller levees, wider crest, armoring, and removal of sand lenses in the repair. The levee includes a seepage cutoff and general levee fill is random generally sandy and silty soil. The geometry used in the analysis is based on recent topographic survey information.

6.4.1 Hydraulic Loading and Consequences

The geotechnical team understands that the flooding consequences that would result from a breach at Winslow Station 29000 are different than flooding as a result of breach at RWDL 495 or Winslow 51500. Figure 12 shows the approximate direction flooding as a result of a levee breach at Winslow Station 29000. Flooding from the index point at 29000 may overlap flooding from point 37000. Economic incremental analysis may help avoid “double counting” damages. The estimated hydraulic loading for Winslow Station 29000 is shown in the third column of Table 6. The main active portion of the channel is not directly adjacent to the levee at this location. River currents used for riverside erosion modeling were assumed to be consistent with the “average channel” flow velocities.

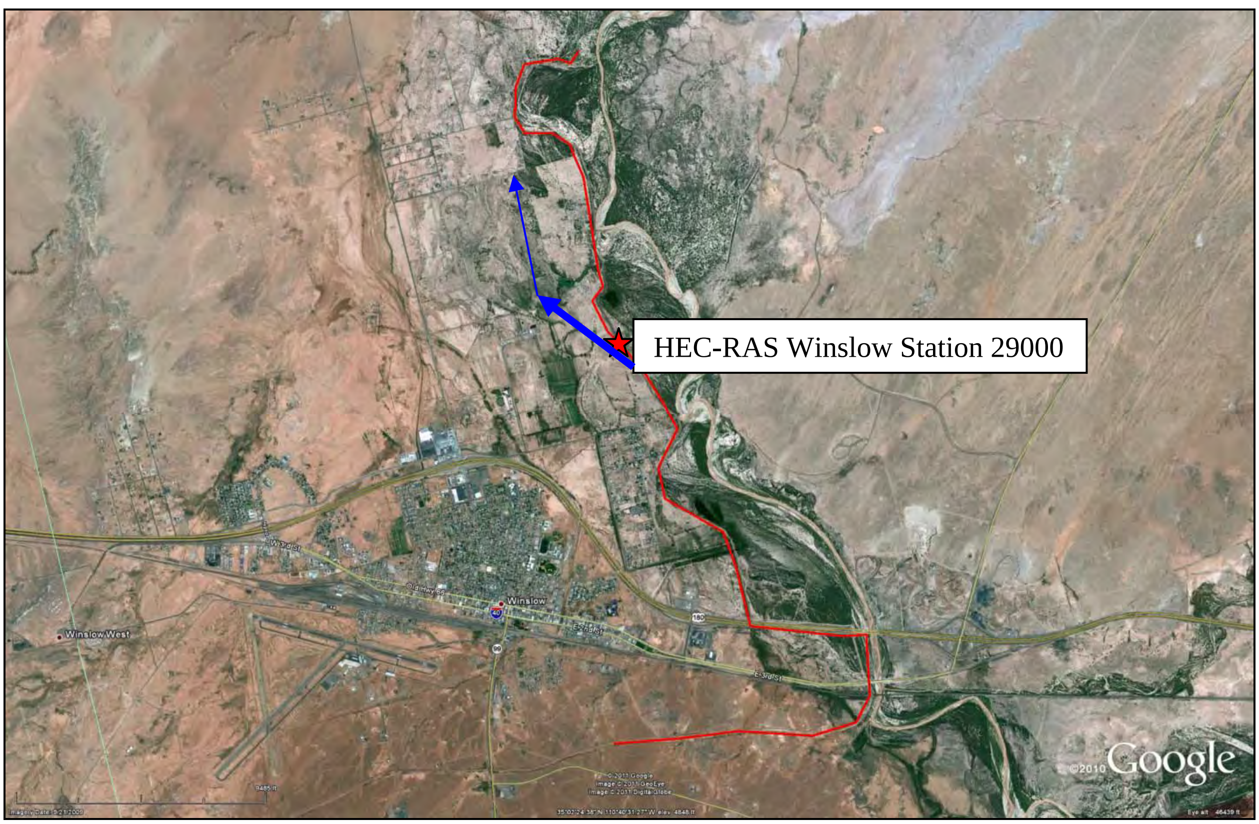


Figure 12. Flood Direction Winslow Station 29000

6.4.2 Past Performance

Due to the meandering of the river, this area of the project has been a significant concern for many and has been observed to have seepage and piping during large events. During 2004 a local rancher discovered a large pipe near this location, and quick response was credited with preventing a levee breach. Rip rap on both sides of the levee has been added, the crest heights have been increased, the crest widths have been increased, and bentonite seepage cutoff has been reconstructed, and channel grading modifications have been performed to try to re-direct the Little Colorado River flow. Photographs of the 2004 event are included in the F3 geotechnical appendix. Seepage has also been noted in previous floods near this location.

6.4.3 Engineering Properties for Evaluation

Winslow Station 29000 has five relevant layers for analysis, a bentonite seepage cutoff, a generally silty sand levee fill, a foundation clay blanket and foundation sands. Below the foundation sands a clayey layer is encountered. These layers are taken from geotechnical exploration stick logs from a 1990s Dames and Moore exploration and shown on the as-built plans. Engineering lab tests were not available at 29000 so literature was used to support the parameters shown below. Figure 13 illustrates the general soil profile for Winslow Station 29000. Graphs 2 and 3 illustrate the basis for engineering parameter selection for the Winslow levees. For the seepage analysis, the absolute values of the hydraulic conductivity are less important than the contrast in permeability between soil layers. Where standard deviations are not shown, the analysis was determined to be insensitive to those parameters. Tables 11 and 12 summarize the material properties used in the engineering analysis. Graph 4 shows the rip-rap gradation tests for the basalt rip-rap along the Winslow Levee.

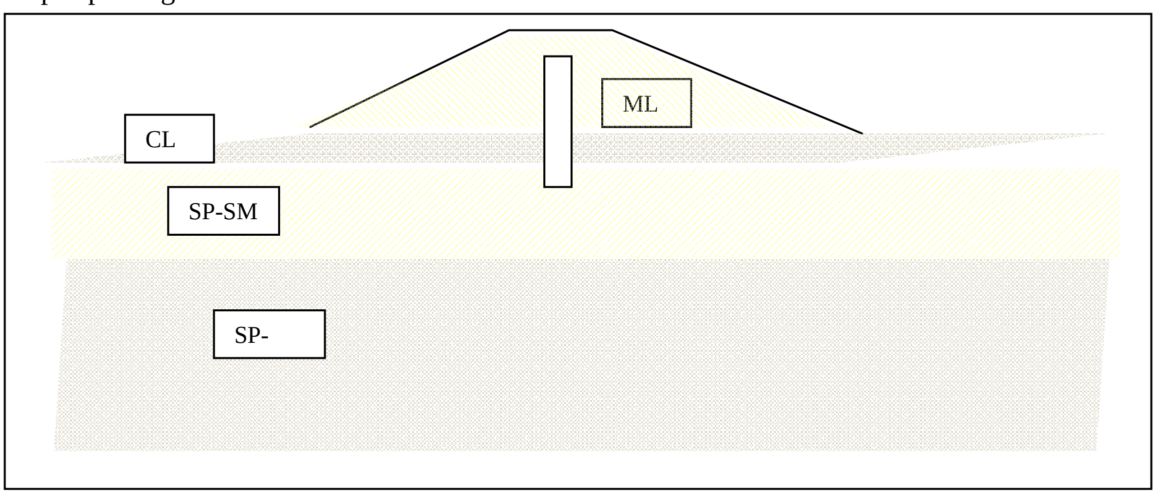


Figure 13. Schematic Cross Section - Winslow Station 29000

Levee Fill: Levee fill was constructed of previous levee fills and channel materials excavated from the nearby channel. The levee fill is generally thought to be silty in nature at this location. The levee was assumed to have somewhat higher strength than the natural deposits because of compaction effort, therefore moderately higher friction angle was warranted. The levee fill is considered highly erodible.

The levee is armored on both sides with approximately 12-inch basalt rip-rap that is anticipated to resist channel velocities in excess of 7 feet/second.

Clayey Foundation Blanket: A very thin clayey blanket was noted in a Dames and Moore stick log nearby Winslow Station 29000. Thin clay blankets are a primary driver of unsatisfactory performance when evaluating potential for pipes, boils and seeps using blanket theory.

Sand Foundation: Near surface soils predominantly consist of sand and silt and the observed levee fill was also sand and silt at the ground surface. Because of the channel nature, it is assumed the soils are in a loose and relatively unconsolidated state. Foundation soils are considered highly erodible.

Bentonite Cutoff: As-built drawings indicate that a bentonite slurry was used to construct a seepage cutoff down the center of the levee. The analysis was most sensitive to the tip elevation of the cutoff and was assumed to have a standard deviation of 2 feet around the expected values.

Table 11. Engineering Properties Used for Stability and Seepage Analysis – Winslow Station 29000

Soil Layer	**Unit Weight (pcf)**	**Vertical Hydraulic Conductivity[7] (cm/sec)**	**Horizontal/ Vertical Conductivity Ratio**	**Effective Friction Angle (degrees)**	**Layer Thickness (ft)**
Bentonite cutoff	95	Less than $1x10^{-5}$	1	22 c= 50 psf	Bottom elev = 4830 ft
Silty Levee Fill (ML)	105 105+σ=115 105-σ=95	$1x10^{-4}$ $1x10^{-4}$+σ=$1x10^{-3}$ $1x10^{-4}$-σ=$1x10^{-5}$	10 10+σ=15 10-σ=4	30 30+σ=32 30-σ=28	-
Foundation Blanket (clayey)	110 110-σ=100 110+σ=120	$1x10^{-4}$ $1x10^{-4}$+σ=$5x10^{-3}$ $1x10^{-4}$-σ=$5x10^{-6}$	10 10+σ=25 10-σ=4	30 30+σ=33 30-σ=27	2 2+σ=3 2-σ=1
Foundation Sand (SP-SM)	115 115-σ=105 115+σ=125	$1x10^{-2}$ $1x10^{-2}$+σ=$1x10^{-1}$ $1x10^{-2}$-σ=$1x10^{-3}$	4 5+σ=10 10-σ=1	34 34+σ=37 34-σ=31	-

Table 12. Property Distribution for Erosion Analysis

Soil Layer Description	Erodibility Coefficient (ft^3/lb-hr)	Critical Stream Velocity
Levee Fill	1.87, coefficient of variation = 0.47	-
Foundation	1.87, coefficient of variation = 0.47	-
Basalt Rip Rap		7 ft/sec

[7] Permeability values are not anticipated to have a Poisson distribution, and are assumed to be approximately log-normally distributed

7.0 Analysis

Each levee section was evaluated for landside slope stability, underseepage, riverside erosion and overtopping erosion as potential failure modes that might cause unsatisfactory performance at the project. At the RWDL 495, consideration was given to the presence of a 36 inch reinforced concrete pipe that penetrates the levee in this location. However, considering field observations did not indicate the presence of any pipe deterioration, the use of robust pipe materials during original construction, and other failure modes (the levee over tops at around a 50-year event), this penetration was not anticipated to contribute significantly to detrimental levee performance. The following sections describe the analysis methods and results and provide the overall probability of unsatisfactory performance at each location evaluated.

Often times in fragility curve analysis, if the engineers performing the analysis do not feel that the analytical tools available capture the full range of potential failure modes that a specific levee may be subject to, additional fragility curves are added using engineering judgment to incorporate additional failure modes. Additional failure modes incorporated often include failure due to rodent burrows, unwanted vegetation, and utility crossings. For this project, the geotechnical team considered the following additional failure modes: failure at utility crossings, failure due to car bodies in the levee, failure due to rodent burrows, failure due to confined sand layers, failure due to dispersive clays, and failure due to unwanted vegetation. These failure modes were considered, and it was judged that additional levee fragility was not warranted. The basis for this judgment is summarized in the following bullet points.

- Utility crossings: There are two known utility crossings for the project, one downstream of RWDL 495 and one through the highway embankment. Although the design and construction details of the utilities are not known, they appeared from exterior visual inspection to be maintained in good condition. Additionally, the pipes appeared to be constructed of reinforced concrete and had suitable outlet structure construction. Piping along utilities is a known phenomenon; however, due to the very short storm events in the area (84 hour storm with 3-4 hour peak. Figure 14), it was judged that there would insufficient time for piping to develop to failure at the crossings. Additionally, the crossing through the highway embankment occurs in a very wide section, and it was judged that probability of failure at that location would be lower than at other potential sections. Additional fragility was not added for these two utility penetrations.

- Car bodies in the levee: Car bodies and non-engineered levee improvements can be a source of potential failure modes. Car bodies were only known to exist in the levee at the very downstream end of the project, and have largely been removed. In addition the levee has been widened and improved at the location and there are minimal consequences if the levee were to fail at this location. An index point was not modeled at this location due to the minimal consequence associated with failure at this location. Because this location was not modeled, additional fragility for car bodies in the levee was not warranted.

- Rodent burrows: Animal burrows may reduce seepage path lengths and cause seepage problems, or can structurally undermine levees if large rodent excavations occur (beaver). Large rodents such as beavers are not anticipated in the area. Small burrows such as caused by ground squirrels were generally not observed. Most of the levee is armored with rip rap on both sides, which generally discourages rodent activity. In addition, the seepage cutoff in the levee is anticipated to generally control seepage performance of the levee, and small decreases in seepage path lengths due to shallow rodent burrows are not anticipated to cause significant seepage problems. The levee performance is not anticipated to be significantly affected by rodent activity

- Confined sand layers: Confined sand layers can cause undesirable build up of pore pressure and high seepage gradients. The entire levee is believed to have a vertical seepage cutoff that is anticipated to dissipate through seepage head. Where landside toe conditions indicate a low permeability layer confining a sand layer, blanket theory analysis was performed to compare seepage gradients to critical gradients of vertical piping. No additional fragility was judged necessary to evaluate confined sands.

- Dispersive clay: A 1980 note by ADWR indicated that an available borrow materials for levee construction were dispersive. The team completing this report believes this assessment in error, and that report may have been intending to rather indicate that the soils are erodible. Other reports (Kleinfelder, 2009) specifically noted that they did not observe significant evidence of dispersive soil. Additional levee fragility due to dispersive clay was not added to the project.

- Unwanted vegetation: Significant vegetation is not observed on the levee. Vegetation observed is not anticipated to have detrimental levee performance effects.

Note that the "weak link" should control an overall levee section, so where two sections were evaluated the section estimated to have worse performance at a given water elevation should be used in the economic analysis. Damages should not be "double counted" if there are overlapping flood maps from the various sections evaluated.

7.1 Seepage Analysis

Seepage analysis was performed using finite difference routines in SEEP/W by GeoSlope International. The primary seepage failure mode considered was underseepage. Through seepage was not considered as a potential mode due to the near continuous presence of a seepage cutoff throughout the project, the low probability of ever achieving the steady state or near steady state conditions required to create an exit face on the downstream slope, and the presence of relatively higher permeability foundation soils that often had clayey blanket layers which could encourage unsatisfactory vertical seepage gradients near the landside toe. In addition, as mentioned previously, underseepage has been documented in that past at Winslow Levee.

The probability of unsatisfactory performance was calculated by determining the distribution of the factor of safety of the vertical exit gradient. The distribution was assumed to have a log-normal shape. The probability of unsatisfactory performance was calculated as the portion of the distribution that resulted in a factor of safety (FS) less than 1.0. Each parameter described in the soil properties section above was varied independently with the other values at the mean expected value, to find the effect of that particular variable on the overall distribution. Using Taylor series, the partial differential equations are solved to determine overall probability of unsatisfactory performance for underseepage. Table 13 below presents the calculated P_u for underseepage at different water stage elevations. All calculations are based on assumed distribution of hydraulic conductivities of the various layers, anisotropic ratios, and that steady state seepage will develop. Boundary conditions were defined as a static water level on the river side, potential seepage face on the landside slope and adjacent grade, and boundary conditions far from the levee equal to the general water table elevations encountered for the area. Table 13 presents the results of the seepage analysis for each levee section at various water elevations.

Table 13. Underseepage Probability of Unsatisfactory Performance

RWDL Station 495		Winslow Station 51500		Winslow Station 37000		Winslow Station 29000	
Water Elevation (ft)	P_u	Water Elevation (ft)	P_u	Water Elevation (ft)	P_u	Water Elevation (ft)	P_u
4866.9	0.04	4865.0	0.05[8]	4853.0	0.01	4849.0	0.82
4864.6	<0.01	4863.2	0.02	4852.5	0	4847.0	0.52
-	-	4861.9	<.01	4850.5	0	4845.0	0.24
-	-	4859.5	0	4849.0	0	4840.0	0
-	-	4855.0	0	-	-	-	-

7.2 Slope Stability Analysis

Slope stability analysis was performed using limit-equilibrium methods contained in the software package Slope/W by GeoSlope International. Expected pore pressures calculated from the seepage analysis were used to evaluate landside slope stability under steady state seepage conditions. The landside stability was analyzed because it is the most likely stability failure mode during a high pool, when potential damages would be the highest. While a rapid drawdown failure of the riverside slope could potentially occur, it is less critical to this analysis because it would occur only after the water has begun to recede. The soil profile was changed 2000 times per water level using a Monte Carlo routine to vary the soil unit weights and strengths described in the engineering parameters above. Circular slip surfaces were used that had a fixed scarp located 10 feet from the riverside crest hinge-point and could exit on the downstream toe area where the minimum factor of safety would occur. This slip surface search criteria was selected as it was judged that a levee crest width less than 10 feet would be considered unsatisfactory performance. A FS less than 1.0 was considered unsatisfactory performance. Table 14 presents the results of the slope stability analysis for each levee section and various water elevations.

Table 14. Slope Stability Probability of Unsatisfactory Performance

Station RWDL 495		Winslow Station 515000		Winslow Station 37000		Winslow Station 29000	
Water Elevation (ft)	P_u	Water Elevation (ft)	P_u	Water Elevation (ft)	P_u	Water Elevation (ft)	P_u
4866.9	0.2495	4865.0	0.035	4853.0	0	4849.0	0.05
4864.6	0.15	4863.0	0.01	4852.5	0	4847.0	0
4862.0	0	4861.0	0	4850.5	0	4845.0	0
-	-	4859.0	0	4849.0	0	4840.0	0

[8] This value was based on shifting the distribution slightly toward unsatisfactory performance calculated at 4863 feet based on the change in expected gradient.

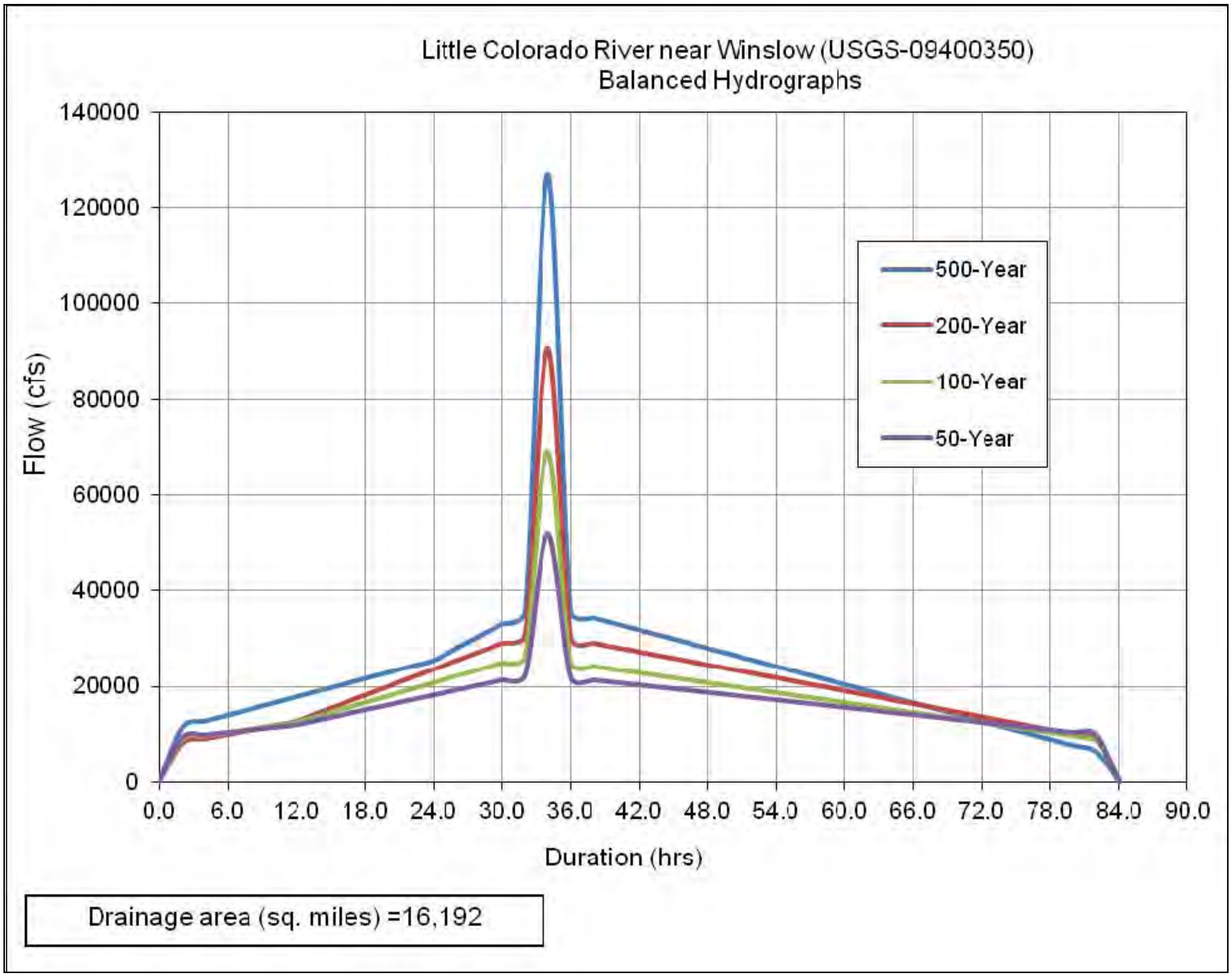
-	-	4855.0	0	-	-	-	-

7.3 Riverside Erosion

Riverside erosion rates were determined using the Excel spread sheet tool written for the Sacramento District of the Army Corps of Engineers by URS. The tool requires an input of the levee slopes, the amount of erosion that would be considered unsatisfactory, channel velocity, duration of loading, loading elevation, channel roughness, foundation erodibility, levee fill erodibility, vegetation condition, armoring critical velocity and channel bend angles and radius. Soil values are randomly selected in 1000 trials, then a range of erosions rates based on the range input parameters is calculated. Erosion rates are calculated using the formula below. The erosion rate is then integrated over the load duration time to determine a total erosion progression for an event.

$$\acute{\varepsilon} = (k\,(\tau - \tau_c))\cdot T$$

where:
k = erodibility coefficient or detachment rate coefficient (ft^3/lb-hr)
τ = effective hydraulic stress on the soil boundary (psf) (function of river velocity)
τ_c = critical shear stress (psf) i.e., the shear stress at which erosion starts (depends on soil properties)
T = Combined parameter of erosion rate adjustment factors (factor to account for vegetation, bends, etc)



(http://waterdata.usgs.gov/usa/nwis/uv?site_no=09400350)

Figure 14. Project Hydrograph for 50 to 500-year Storm Events

For this project, the average flow for 84 hours was used to determine channel velocities for erosion. The elevation used to combine the erosion failure mode with the other modes was the peak storm elevation, although the average elevation during the 84 hours and modeled in the erosion analysis is much lower (see Figure 14). Where the main river flow channel was located adjacent to the levee, the main channel flows provided by H&H were used, otherwise average channel velocities were used. In general, where the levee was armored on the riverside, the primary erosion driver was foundation erosion. Table 15 summarizes the results of the riverside erosion analysis for each levee section evaluated.

Table 15. Riverside Erosion Probability of Unsatisfactory Performance

Station RWDL 495		Winslow Station 51500[9]		Winslow Station 37000		Winslow Station 29000	
Water Elevation (ft)	P_u	Water Elevation (ft)	P_u	Water Elevation (ft)	P_u	Water Elevation (ft)	P_u
4866.9	0	4865.0	0.025	4853.0	0.13	4849.0	0
4864.6	0	4863.0	0.015	4852.5	0.08	4847.0	0
4862.0	0	4861.0	0.01	4850.5	0.02	4845.0	0
-	-	4859.0	0.004	4849.0	0.001	4840.0	0
-	-	4855.0	0	-	-	4849.0	0

7.4 Overtopping Erosion

Overtopping erosion was assumed to cause failure of unarmored levee sections almost instantly, as the levee fill is very highly erodible, as demonstrated by significant rilling and other erosion from just rainfall. For the armor sections, a shear stress by sheet flow of 1 to 2 feet of water was calculated as the force of the water that would act parallel to the slope, based on the gravitational force of the water that was parallel slope angle. The landside rip rap was estimated to have a 50 percent chance of withstanding the flow for water levels exceeding the crest by about 2 feet. The overtopping erosion estimates are highly based on engineering judgment. Table 16 summarizes the estimate of overtopping erosion probability of unsatisfactory performance.

Table 16. Overtopping Erosion Probability of Unsatisfactory Performance

Station RWDL 495		Winslow Station 51500		Winslow Station 37000		Winslow Station 29000	
Water Elevation (ft)	P_u	Water Elevation (ft)	P_u	Water Elevation (ft)	P_u	Water Elevation (ft)	P_u
4867.9	1	4866.0	1	4857.2	0.5	4852.0	0.5
4866.9	0	4865	0	4856.2	0.25	4851.0	0.25
4862.0	0	4861.0	0	4855.0	0	4850.0	0
-	-	4859.0	0	4849.0	0	4840.0	0
-	-	4855.0	0	-	-	4849.0	0

[9] The elevations that Pu was calculated are slightly different than reported in appendix C. The Pu for the reported elevations was interpolated from the Pu calculated at elevations shown in Appendix C. This was to match the elevations where stability and seepage were calculated for combining curves. A change in H&H data led to changing the erosion calculations (at elevations corresponding with standard return periods) after slope stability and seepage was already completed.

8.0 Combined Probability of Unsatisfactory Performance

The combined overall probability of Unsatisfactory Performance is determined by combining each of the previously described failure modes. The overall Pu at each elevation is calculated by the formula;

$$P_u(\text{elev.})=1-(1-P_{uus})(1-P_{uss})(1-P_{uwse})(1-P_{uote})$$

where:
P_u(elev.)= combined probability of failure for a given water surface elevation
P_{uus} = P_u Underseepage
P_{uss} = P_u Slope Stability
P_{uwse} = P_u Riverside Erosion
P_{uote}= P_u Overtopping Erosion

The overall probability of unsatisfactory performance does not absolutely indicate the probability of a catastrophic levee breach. The P_u values represent the probability that there are ground conditions near the referenced Station where, if the water loading elevation indicated is reached, an underseepage or landside slope stability factor of safety is reduced below 1.0 or an erosion progression that results in a crest width less than 10 feet will occur. Care should be taken in application of the P_u values in the economics as to avoid double counting damages due to overlapping flood plains. The "weak link" for each economic area should be chosen to calculate damages for the without project condition.

This analysis is intended only for feasibility-level analysis. The values are not design values for new structures, and appropriate exploration, lab testing and engineering analysis should be performed for new project design. New project designs may wish to incorporate features that improve reliability, however, these values are not intended to be used to support or specifically target levee design. Table 17 presents in tabular form the estimated P_u for different water elevations for each section evaluated. Figure 15 shows the probability of unsatisfactory performance and approximate return period for different water elevations for the sections below.

Table 17. Combined Probability of Unsatisfactory Performance

RWDL Section 495		Section 51500		Section 37000		Section 29000	
Water Elevation (ft)	P_u	Water Elevation (ft)	P_u	Water Elevation (ft)	P_u	Water Elevation (ft)	P_u
4868.0	1.0	4866.0	1.00	4857.0	0.50	4852.0	1
4866.9	0.31	4865.0	0.11	4856.0	0.25	4849.0	0.83
4864.6	0.16	4863.0	0.04	4853.0	0.14	4847.0	0.52
4862.0	0	4862.0	0.02	4852.5	0.08	4845.0	0.24
-	-	4859.5	0.00	4850.5	0.02	4840.0	0
-	-	4855.0	0.00	4849.0	0.001	-	-

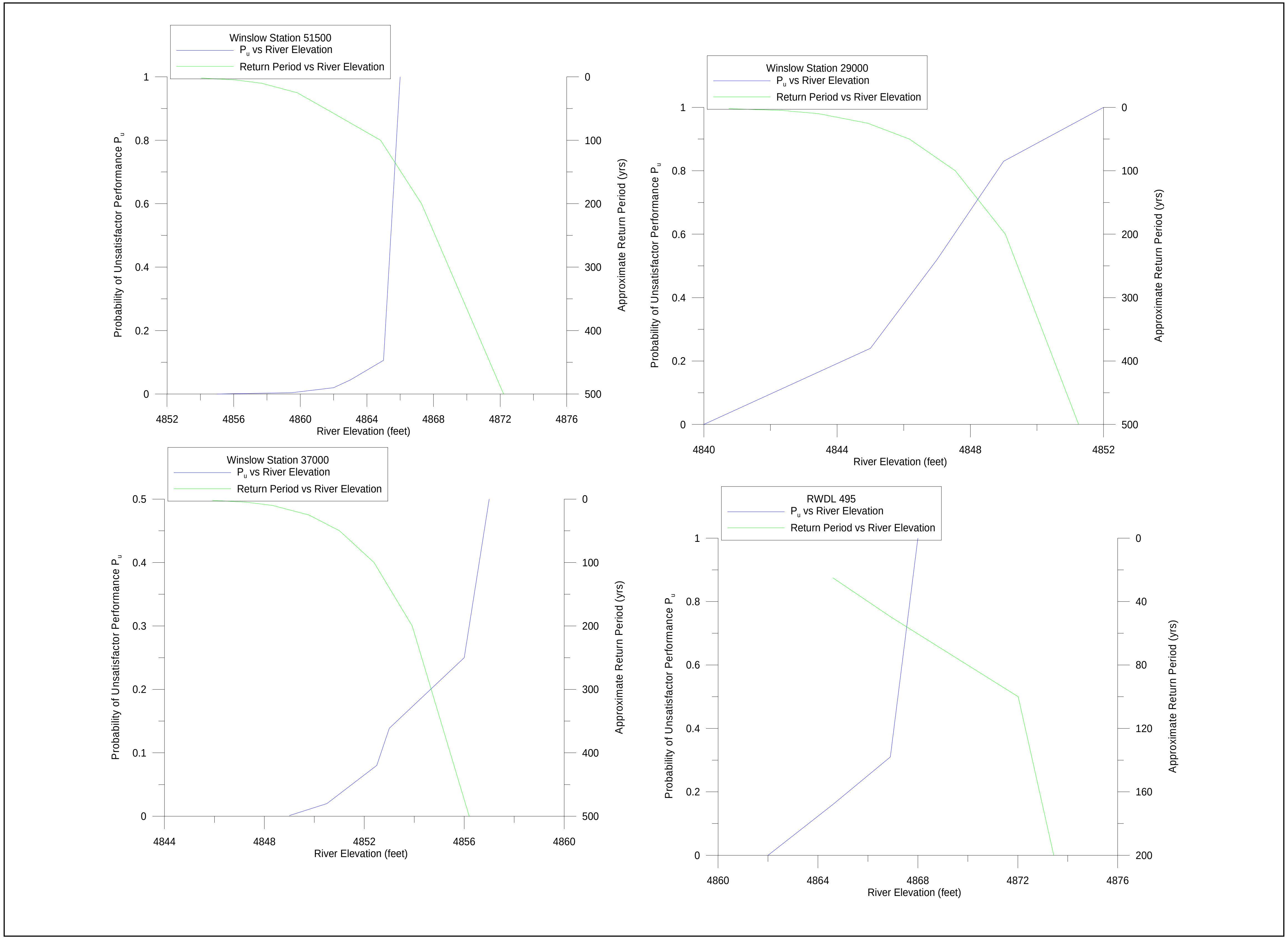


Figure 15. Combined Probability of Unsatisfactory Performance

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Appendix A: Underseepage Calculation Summaries

The following Tables summarize the underseepage calculations for the RWDL and Winslow Levee sections evaluated. In general, important parameters were varied one standard deviation around the mean to calculate the vertical gradient using SEEP/W software. The results were tabularized, and a lognormal distribution of the calculated factor of safety was calculated, as illustrated in ETL 1110-2-556. Since hundreds of SEEP/W model runs were performed, only one example output from SEEP/W is shown with the associated table summaries of all of the seepage runs used in the analysis prepared for this report.

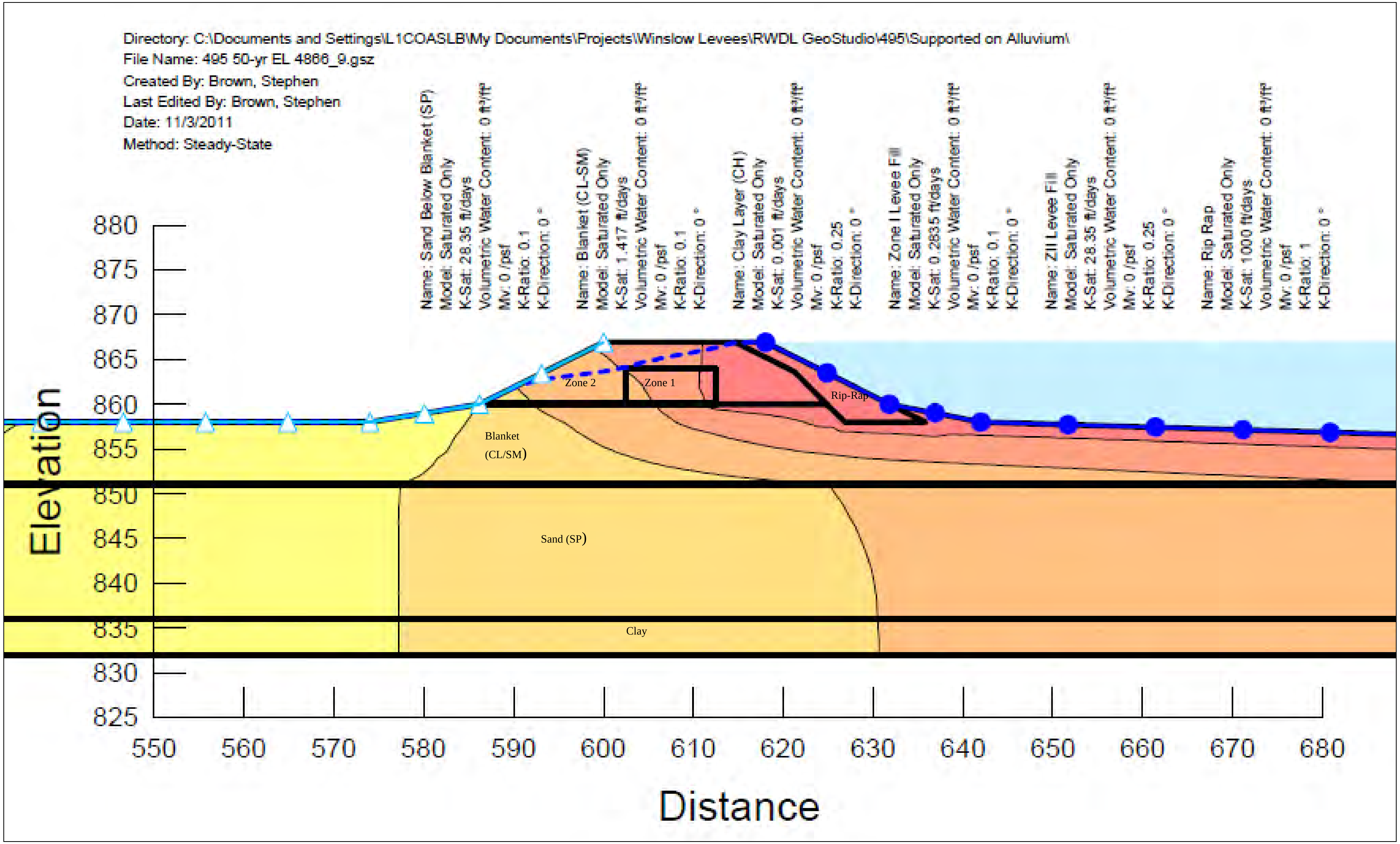


Figure A-1 Example Seepage Calculation for RWDL495 (units are in feet).

	Zone 1				Zone2				Unit weight	Moenkopi Sandstone			Critical Gradient	Calculated Gradient	Factor of Safety	
	kv	kh/kv	$(kh/kv)^{-1}$	kh	kv	kh/kv	$(kh/kv)^{-1}$	kh		kv	kh/kv	kh				
	fpd			fpd	fpd			fpd	pcf	fpd		fpd		(from SEEP/W)		
Case 1	0.283	10.00	0.10	2.8	28.35	4.00	0.25	113.4	125	0.0002835	25	0.007087	1.00	0.68	1.962151207	
Case 2	2.835	10.00	0.10	28.3	28.35	4.00	0.25	113.4	125	0.0002835	25	0.007087	1.00	0.684	1.950676638	
Case 3	0.028	10.00	0.10	0.3	28.35	4.00	0.25	113.4	125	0.0002835	25	0.007087	1.00	0.679	1.965040973	
Case 4	0.283	4.00	0.25	1.1	28.35	4.00	0.25	113.4	125	0.0002835	25	0.007087	1.00	0.68	1.962151207	
Case 5	2.835	4.00	0.25	11.3	28.35	4.00	0.25	113.4	125	0.0002835	25	0.007087	1.00	0.685	1.947828935	
Case 6	0.028	4.00	0.25	0.1	28.35	4.00	0.25	113.4	125	0.0002835	25	0.007087	1.00	0.68	1.962151207	
Case 7	0.283	25.00	0.04	7.1	28.35	4.00	0.25	113.4	125	0.0002835	25	0.007087	1.00	0.68	1.962151207	
Case 8	2.835	25.00	0.04	70.9	28.35	4.00	0.25	113.4	125	0.0002835	25	0.007087	1.00	0.684	1.950676638	
Case 9	0.028	25.00	0.04	0.7	28.35	4.00	0.25	113.4	125	0.0002835	25	0.007087	1.00	0.679	1.965040973	4.68709E-05
Case 10	0.283	10.00	0.10	2.8	28.35	4.00	0.25	113.4	125	0.0002835	25	0.007087	1.00	0.68	1.962151207	
Case 11	0.283	10.00	0.10	2.8	283.46	4.00	0.25	1133.9	125	0.0002835	25	0.007087	1.00	0.677	1.970846116	
Case 12	0.283	10.00	0.10	2.8	2.83	4.00	0.25	11.3	125	0.0002835	25	0.007087	1.00	0.684	1.950676638	
Case 13	0.283	10.00	0.10	2.8	28.35	10.00	0.10	283.5	125	0.0002835	25	0.007087	1.00	0.664	2.009431959	
Case 14	0.283	10.00	0.10	2.8	283.46	10.00	0.10	2834.6	125	0.0002835	25	0.007087	1.00	0.663	2.012462776	
Case 15	0.283	10.00	0.10	2.8	2.83	10.00	0.10	28.3	125	0.0002835	25	0.007087	1.00	0.669	1.994413783	
Case 16	0.283	10.00	0.10	2.8	28.35	1.00	1.00	28.3	125	0.0002835	25	0.007087	1.00	0.691	1.930915804	
Case 17	0.283	10.00	0.10	2.8	283.46	1.00	1.00	283.5	125	0.0002835	25	0.007087	1.00	0.687	1.9421584	
Case 18	0.283	10.00	0.10	2.8	2.83	1.00	1.00	2.8	125	0.0002835	25	0.007087	1.00	0.696	1.917044282	0.001164983
Case 19	0.283	10.00	0.10	2.8	28.35	4.00	0.25	113.4	125	0.0002835	25	0.007087	1.00	0.68	1.962151207	
Case 20	0.283	10.00	0.10	2.8	28.35	4.00	0.25	113.4	115	0.0002835	25	0.007087	0.84	0.68	1.648708522	
Case 21	0.283	10.00	0.10	2.8	28.35	4.00	0.25	113.4	135	0.0002835	25	0.007087	1.16	0.68	2.275593891	0.098246317
Case 22	0.283	10.00	0.10	2.8	28.35	4.00	0.25	113.4	125	0.0002835	25	0.007087	1.00	0.68	1.962151207	
Case 23	0.283	10.00	0.10	2.8	28.35	4.00	0.25	113.4	125	0.0002835	10	0.002835	1.00	0.537	2.484660746	
Case 24	0.283	10.00	0.10	2.8	28.35	4.00	0.25	113.4	125	0.0002835	100	0.028346	1.00	0.79	1.688940279	0.163471915

Expected Gradient	0.68	Variance	0.26
Expected Factor of Safety	1.96	Std Dev	0.512767087
Expected FS	1.962		
V=	0.26		
sigmalnFS	0.26		
E(lnFS)	0.657064068		
beta	2.55641946		
FS critical = 1	1		
ln (FS crit)	0		
Z=	-2.55641946		
Pu=	**0.005287777**		

Critical Gradient = $(\gamma-\gamma_w)/\gamma_w$

Calculated Gradient:

(NO blanket layer) = Max +Y-gradient at toe of landside slope using Seep/W output

(WITH blanket layer) = (Max. ΔHead across blanket layer)/blanket thickness

Table A-1. Station RWDL 495 Water Elevation 4866.9, Sandstone Foundation Calculated Gradients, and P_u

	Zone 1				Zone2				Blanket						Sand Below blanket			Critical Gradient	Total Head at Bottom of Blanket (ft)	Total Head at Top of Blanket (ft)	Calculated Gradient (from SEEP/W)	Factor of Safety	
	kv	kh/kv	$(kh/kv)^{-1}$	kh	kv	kh/kv	$(kh/kv)^{-1}$	kh	thickness	unit weight	kv	kh/kv	$(kh/kv)^{-1}$	kh	kv	kv/kh	kh						
	fpd			fpd	fpd			fpd	ft	pcf	fpd			fpd	fpd		fpd						
Case 1	0.283	10	0.10	2.8	28.35	4	0.25	113.4	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.908	858	0.27	2.504636349	
Case 2	2.835	10	0.10	28.3	28.35	4	0.25	113.4	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.919	858	0.27	2.490279392	
Case 3	0.028	10	0.10	0.3	28.35	4	0.25	113.4	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.906	858	0.27	2.507264509	
Case 4	0.283	4	0.25	1.1	28.35	4	0.25	113.4	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.909	858	0.27	2.503324334	
Case 5	2.835	4	0.25	11.3	28.35	4	0.25	113.4	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.921	858	0.27	2.487686702	
Case 6	0.028	4	0.25	0.1	28.35	4	0.25	113.4	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.906	858	0.27	2.507264509	
Case 7	0.283	25	0.04	7.1	28.35	4	0.25	113.4	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.908	858	0.27	2.504636349	
Case 8	2.835	25	0.04	70.9	28.35	4	0.25	113.4	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.918	858	0.27	2.491577765	
Case 9	0.028	25	0.04	0.7	28.35	4	0.25	113.4	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.906	858	0.27	2.507264509	6.6E-05
Case 10	0.283	10	0.10	2.8	28.35	4	0.25	113.4	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.908	858	0.27	2.504636349	
Case 11	0.283	10	0.10	2.8	283.46	4	0.25	1133.9	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.909	858	0.27	2.503324334	
Case 12	0.283	10	0.10	2.8	2.83	4	0.25	11.3	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.878	858	0.27	2.544646514	
Case 13	0.283	10	0.10	2.8	28.35	10	0.10	283.5	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.887	858	0.27	2.532509885	
Case 14	0.283	10	0.10	2.8	283.46	10	0.10	2834.6	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.891	858	0.27	2.527152911	
Case 15	0.283	10	0.10	2.8	2.83	10	0.10	28.3	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.854	858	0.26	2.577586922	
Case 16	0.283	10	0.10	2.8	28.35	1	1.00	28.3	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.925	858	0.27	2.482517483	
Case 17	0.283	10	0.10	2.8	283.46	1	1.00	283.5	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.922	858	0.27	2.48639238	
Case 18	0.283	10	0.10	2.8	2.83	1	1.00	2.8	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.904	858	0.27	2.50989819	0.000908
Case 19	0.283	10	0.10	2.8	28.35	4	0.25	113.4	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.908	858	0.27	2.504636349	
Case 20	0.283	10	0.10	2.8	28.35	4	0.25	113.4	5	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	860.156	858	0.43	1.58323819	
Case 21	0.283	10	0.10	2.8	28.35	4	0.25	113.4	9	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.748	858	0.19	3.515006161	0.933592
Case 22	0.283	10	0.10	2.8	28.35	4	0.25	113.4	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.908	858	0.27	2.504636349	
Case 23	0.283	10	0.10	2.8	28.35	4	0.25	113.4	7	90.00	1.42	10	0.10	14.2	28.35	10	283.5	0.442307692	859.908	858	0.27	1.622722142	
Case 24	0.283	10	0.10	2.8	28.35	4	0.25	113.4	7	120.00	1.42	10	0.10	14.2	28.35	10	283.5	0.923076923	859.908	858	0.27	3.386550556	0.777773
Case 25	0.283	10	0.10	2.8	28.35	4	0.25	113.4	7	105.00	1.42	10	0.10	14.2	28.35	10	283.5	0.682692308	859.908	858	0.27	2.504636349	
Case 26	0.283	10	0.10	2.8	28.35	4	0.25	113.4	7	105.00	14.17	10	0.10	141.7	28.35	10	283.5	0.682692308	860.416	858	0.35	1.977999236	
Case 27	0.283	10	0.10	2.8	28.35	4	0.25	113.4	7	105.00	0.01	10	0.10	0.1	28.35	10	283.5	0.682692308	-	-			
Case 28	0.283	10	0.10	2.8	28.35	4	0.25	113.4	7	105.00	1.42	4	0.25	5.7	28.35	10	283.5	0.682692308	860.529	858	0.36	1.889618883	
Case 29	0.283	10	0.10	2.8	28.35	4	0.25	113.4	7	105.00	14.17	4	0.25	56.7	28.35	10	283.5	0.682692308	859.886	858	0.27	2.53385268	
Case 30	0.283	10	0.10	2.8	28.35	4	0.25	113.4	7	105.00	0.01	4	0.25	0.1	28.35	10	283.5	0.682692308	-	-			
Case 31	0.283	10	0.10	2.8	28.35	4	0.25	113.4	7	105.00	1.42	25	0.04	35.4	28.35	10	283.5	0.682692308	858.388	858			
Case 32	0.283	10	0.10	2.8	28.35	4	0.25	113.4	7	105.00	14.17	25	0.04	354.3	28.35	10	283.5	0.682692308	860.655	858	0.38	1.799942054	
Case 33	0.283	10	0.10	2.8	28.35	4	0.25	113.4	7	105.00	0.01	25	0.04	0.4	28.35	10	283.5	0.682692308	-	-			0.123162

Expected Gradient	0.27	Variance	1.8355
Expected Factor of Safety	2.50	Std Dev	1.354806
Expected FS	2.505		
V=	0.54		
sigmalnFS	0.51		
E(lnFS)	0.865549246		
beta	1.708521303		
FS critical = 1	1		
ln (FS crit)	0		
Z=	-1.708521303		
Pu=	**0.043769829**		

Critical Gradient = $(\gamma-\gamma_w)/\gamma_w$

Calculated Gradient:

(NO blanket layer) = Max +Y-gradient at toe of landside slope using Seep/W output

(WITH blanket layer) = (Max. ΔHead across blanket layer)/blanket thickness

Table A-2. Station RWDL 495 Water Elevation 4866.9, Alluvium Foundation Calculated Gradients, and P_u

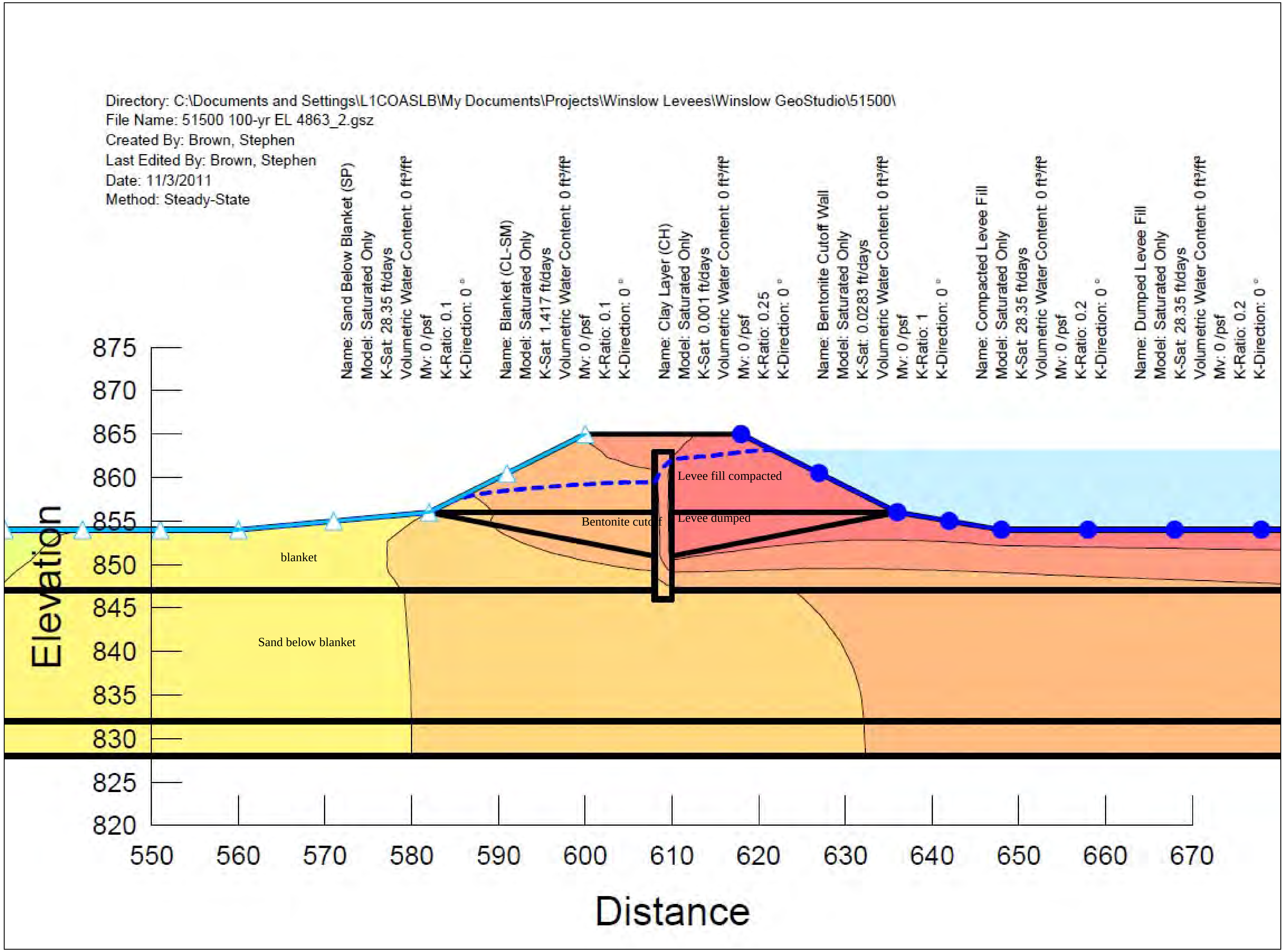


Figure A-2 Example Seepage Calculation for Winslow Station 51500 (units are in feet).

	Levee Fill kv fpd	kh/kv	$(kh/kv)^{-1}$	kh fpd	Bentonite cuttoff depth bottom elevation ft	kv=kh fpd	Blanket thickness ft	unit weight pcf	kv fpd	kh/kv	$(kh/kv)^{-1}$	kh fpd	Sand Below blanket kv fpd	kv/kh	kh fpd	Critical Gradient	Total Head at Bottom of Blanket (ft)	Total Head at Top of Blanket (ft)	Calculated Gradient (from SEEP/W)	Factor of Safety	
Case 1	28.35	5.00	0.20	141.7	4846	0.0283	7	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	855.16	854	0.17	4.11969496	
Case 2	283.46	5.00	0.20	1417.3	4846	0.0283	7	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	855.189	854	0.17	4.019214595	0.000688367
Case 3	2.83	5.00	0.20	14.2	4846	0.0283	7	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	854.901	854	0.13	5.303935798	
Case 4	28.35	2.00	0.50	56.7	4846	0.0283	7	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	855.212	854	0.17	3.942942371	
Case 5	283.46	2.00	0.50	566.9	4846	0.0283	7	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	855.24	854	0.18	3.853908189	
Case 6	2.83	2.00	0.50	5.7	4846	0.0283	7	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	855.011	854	0.14	4.726850795	
Case 7	28.35	25.00	0.04	708.7	4846	0.0283	7	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	854.963	854	0.14	4.962457065	
Case 8	283.46	25.00	0.04	7086.6	4846	0.0283	7	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	855.042	854	0.15	4.586224716	
Case 9	2.83	25.00	0.04	70.9	4846	0.0283	7	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	854.669	854	0.10	7.143267793	
Case 10	28.35	5.00	0.20	141.7	4846	0.0283	7	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	855.16	854	0.17	4.11969496	
Case 11	28.35	5.00	0.20	141.7	4844	0.0283	7	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	855.087	854	0.16	4.396362607	3.94762E-05
Case 12	28.35	5.00	0.20	141.7	4848	0.0283	7	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	855.166	854	0.17	4.098495844	
Case 13	28.35	5.00	0.20	141.7	4846	0.0283	7	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	855.16	854	0.17	4.11969496	
Case 14	28.35	5.00	0.20	141.7	4846	0.0283	5	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	855.417	854	0.28	2.408935454	0.008569592
Case 15	28.35	5.00	0.20	141.7	4846	0.0283	9	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	854.907	854	0.10	6.774234586	
Case 16	28.35	5.00	0.20	141.7	4846	0.0283	7	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	855.16	854	0.17	4.11969496	
Case 17	28.35	5.00	0.20	141.7	4846	0.0283	7	90.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.44	855.16	854	0.17	2.669098143	0
Case 18	28.35	5.00	0.20	141.7	4846	0.0283	7	120.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.92	855.16	854	0.17	5.570291777	
Case 19	28.35	5.00	0.20	141.7	4846	0.0283	7	105.00	1.417	10.00	0.10	14.17	28.35	10.00	283.5	0.68	855.16	854	0.17	4.11969496	
Case 20	28.35	5.00	0.20	141.7	4846	0.0283	7	105.00	14.173	10.00	0.10	141.73	28.35	10.00	283.5	0.68	855.673	854	0.24	2.85645317	0.002568503
Case 21	28.35	5.00	0.20	141.7	4846	0.0283	7	105.00	0.014	10.00	0.10	0.14	28.35	10.00	283.5	0.68	-	-			
Case 22	28.35	5.00	0.20	141.7	4846	0.0283	7	105.00	1.417	4.00	0.25	5.67	28.35	10.00	283.5	0.68	855.894	854	0.27	2.523150028	
Case 23	28.35	5.00	0.20	141.7	4846	0.0283	7	105.00	14.173	4.00	0.25	56.69	28.35	10.00	283.5	0.68	855.171	854	0.17	4.080995862	
Case 24	28.35	5.00	0.20	141.7	4846	0.0283	7	105.00	0.014	4.00	0.25	0.06	28.35	10.00	283.5	0.68	-	-			
Case 25	28.35	5.00	0.20	141.7	4846	0.0283	7	105.00	1.417	25.00	0.04	35.43	28.35	10.00	283.5	0.68	-	-			
Case 26	28.35	5.00	0.20	141.7	4846	0.0283	7	105.00	14.173	25.00	0.04	354.33	28.35	10.00	283.5	0.68	855.95	854	0.28	2.450690335	
Case 27	28.35	5.00	0.20	141.7	4846	0.0283	7	105.00	0.014	25.00	0.04	0.35	28.35	10.00	283.5	0.68	-	-			0.011865939

Expected Gradient	0.17	Variance	0.023731878
Expected Factor of Safety	4.12	Std Dev	0.154051545
Expected FS	4.120		
V=	0.71		
sigmalnFS	0.64		
E(lnFS)	1.364727606		
beta	2.131146953		
FS critical = 1	1		
ln (FS crit)	0		
Z=	-2.131146953		
Pu=	**0.016538519**		

Critical Gradient = $(\gamma-\gamma_w)/\gamma_w$

Calculated Gradient:

(NO blanket layer) = Max +Y-gradient at toe of landside slope using Seep/W output

(WITH blanket layer) = (Max. ΔHead across blanket layer)/blanket thickness

Table A-3 Winslow Station 51500, Water Surface Elevation 4863 feet

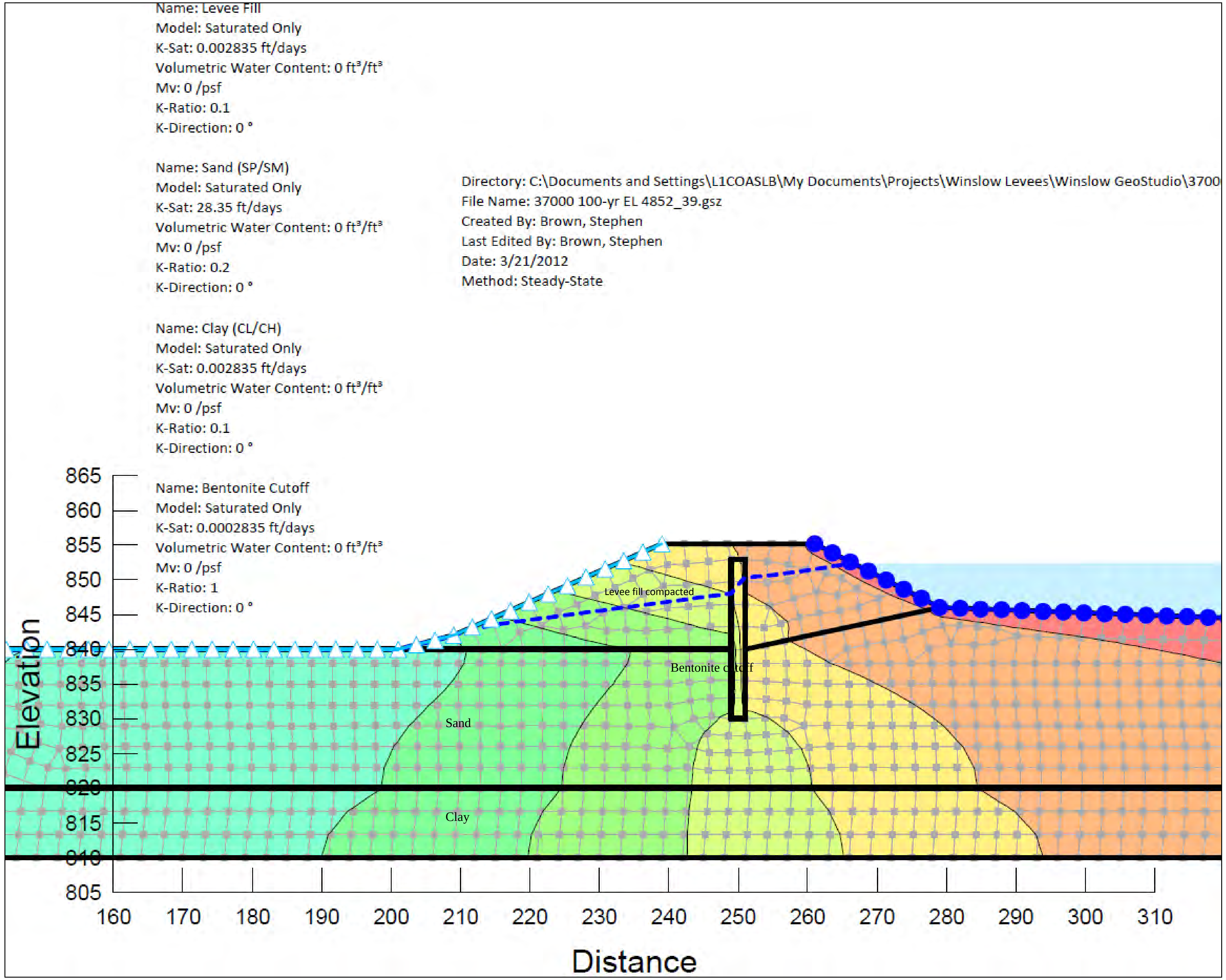


Figure A-3 Example Seepage Calculation for Winslow Station 37000 (units are in feet).

	Levee Fill				Bentonite cuttoff		Sand					Clay below sand			Critical Gradient	Calculated Gradien	Factor of Safety	
	kv	kh/kv	$(kh/kv)^{-1}$	kh		kv=kh	gamma	kv	kv/kh	$(kh/kv)^{-1}$	kh	kv	kh/kv	kh				
	fpd			fpd		cm/sec	pcf	fpd			fpd	fpd		fpd		(from SEEP/W)		
Case 1	0.00283	10.00	0.10	0.02835	4830	0.00028	110	28.35	5.00	0.20	141.7	0.002835	10.00	0.0283	0.762820513	0.146	5.224798033	
Case 2	0.02835	10.00	0.10	0.28346	4830	0.00028	110	28.35	5.00	0.20	141.7	0.002835	10.00	0.0283	0.762820513	0.146	5.224798033	
Case 3	0.00028	10.00	0.10	0.00283	4830	0.00028	110	28.35	5.00	0.20	141.7	0.002835	10.00	0.0283	0.762820513	0.146	5.224798033	
Case 4	0.00283	5.00	0.20	0.01417	4830	0.00028	110	28.35	5.00	0.20	141.7	0.002835	10.00	0.0283	0.762820513	0.146	5.224798033	
Case 5	0.02835	5.00	0.20	0.14173	4830	0.00028	110	28.35	5.00	0.20	141.7	0.002835	10.00	0.0283	0.762820513	0.146	5.224798033	
Case 6	0.00028	5.00	0.20	0.00142	4830	0.00028	110	28.35	5.00	0.20	141.7	0.002835	10.00	0.0283	0.762820513	0.146	5.224798033	
Case 7	0.00283	25.00	0.04	0.07087	4830	0.00028	110	28.35	5.00	0.20	141.7	0.002835	10.00	0.0283	0.762820513	0.146	5.224798033	
Case 8	0.02835	25.00	0.04	0.70866	4830	0.00028	110	28.35	5.00	0.20	141.7	0.002835	10.00	0.0283	0.762820513	0.146	5.224798033	
Case 9	0.00028	25.00	0.04	0.00709	4830	0.00028	110	28.35	5.00	0.20	141.7	0.002835	10.00	0.0283	0.762820513	0.146	5.224798033	0
Case 10	0.00283	10.00	0.10	0.02835	4830	0.00028	110	28.35	5.00	0.20	141.7	0.002835	10.00	0.0283	0.762820513	0.146	5.224798033	
Case 11	0.00283	10.00	0.10	0.02835	4830	0.00028	100	28.35	5.00	0.20	141.7	0.002835	10.00	0.0283	0.602564103	0.146	4.127151387	
Case 12	0.00283	10.00	0.10	0.02835	4830	0.00028	120	28.35	5.00	0.20	141.7	0.002835	10.00	0.0283	0.923076923	0.146	6.322444679	1.204828
Case 13	0.00283	10.00	0.10	0.02835	4830	0.00028	110	28.35	5.00	0.20	141.7	0.002835	10.00	0.0283	0.762820513	0.146	5.224798033	
Case 14	0.00283	10.00	0.10	0.02835	4830	0.00028	110	283.46	5.00	0.20	1417.3	0.002835	10.00	0.0283	0.762820513	0.146	5.224798033	
Case 15	0.00283	10.00	0.10	0.02835	4830	0.00028	110	2.83	5.00	0.20	14.2	0.002835	10.00	0.0283	0.762820513	0.146	5.224798033	
Case 16	0.00283	10.00	0.10	0.02835	4830	0.08100	110	28.35	2.00	0.50	56.7	0.002835	10.00	0.0283	0.762820513	0.081	9.417537195	
Case 17	0.00283	10.00	0.10	0.02835	4830	0.00028	110	283.46	2.00	0.50	566.9	0.002835	10.00	0.0283	0.762820513	0.081	9.417537195	
Case 18	0.00283	10.00	0.10	0.02835	4830	0.00028	110	2.83	2.00	0.50	5.7	0.002835	10.00	0.0283	0.762820513	0.081	9.417537195	
Case 19	0.00283	10.00	0.10	0.02835	4830	0.00028	110	28.35	10.00	0.10	283.5	0.002835	10.00	0.0283	0.762820513	0.211	3.615263094	
Case 20	0.00283	10.00	0.10	0.02835	4830	0.00028	110	283.46	10.00	0.10	2834.6	0.002835	10.00	0.0283	0.762820513	0.211	3.615263094	
Case 21	0.00283	10.00	0.10	0.02835	4830	0.00028	110	2.83	10.00	0.10	28.3	0.002835	10.00	0.0283	0.762820513	0.211	3.615263094	6.729506
															Expected Gradient	0.15	Variance	7.934334
															Expected Factor of Safety	5.22	Std Dev	2.816795
																5.220		
															V=	0.54		
															sigmalnFS	0.51		
															E(lnFS)	1.627754479		
															beta	3.222508234		
															FS critical = 1	1		
															ln (FS crit)	0		
															Z=	-3.222508234		
															Pu=	**0.000635368**		

Critical Gradient = $(\gamma-\gamma_w)/\gamma_w$

Calculated Gradient:

(NO blanket layer) = Max +Y-gradient at toe of landside slope using Seep/W output

(WITH blanket layer) = (Max. ΔHead across blanket layer)/blanket thickness

Table A-4 Winslow Station 37000, Water Surface Elevation 4852 feet

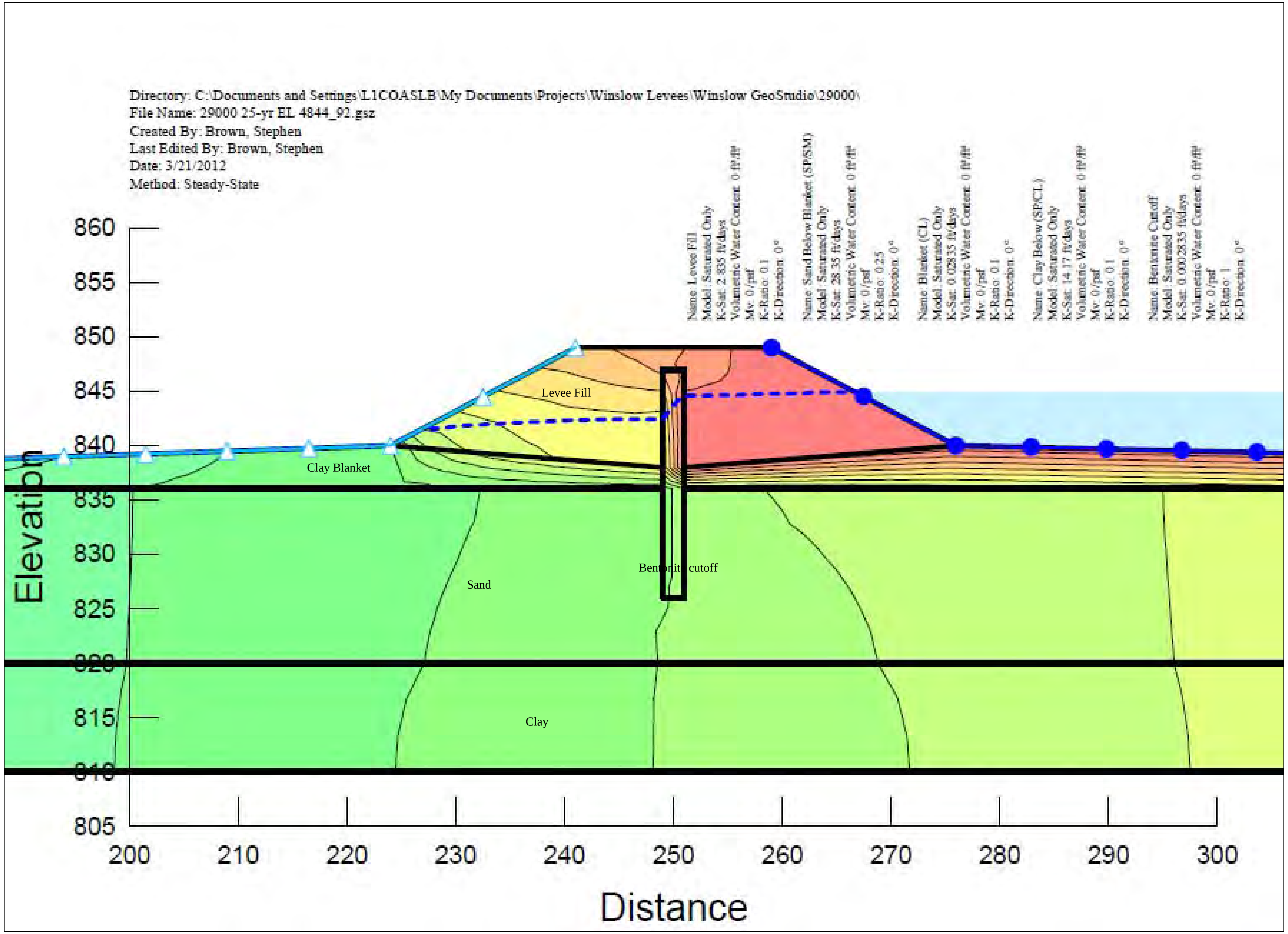


Figure A-4 Example Seepage Calculation for Winslow Station 29000 (units are in feet).

	Levee Fill				Bentonite cutoff		Blanket						Sand Below blanket				SP-CL below sand				Critical Gradient	Calculated Gradien	Factor of Safety	
	kv	kh/kv	$(kh/kv)^{-1}$	kh		kv=kh	thickness	unit weight	kv	kh/kv	$(kh/kv)^{-1}$	kh	kv	kv/kh		kh	kv	kh/kv	$(kh/kv)^{-1}$	kh				
	ft/day			ft/day		ft/day	ft	pcf	ft/day			ft/day	ft/day			ft/day	ft/day			ft/day		(from SEEP/W)		
Case 1	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 2	2.83465	10.00	0.10	28.34646	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 3	0.02835	10.00	0.10	0.28346	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 4	0.28346	4.00	0.25	1.13386	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 5	2.83465	4.00	0.25	11.33858	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 6	0.02835	4.00	0.25	0.11339	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 7	0.28346	15.00	0.07	4.25197	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 8	2.83465	15.00	0.07	42.51969	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 9	0.02835	15.00	0.07	0.42520	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 10	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	0
Case 11	0.28346	10.00	0.10	2.83465	4830	0.00028	1	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.9	0.847578348	
Case 12	0.28346	10.00	0.10	2.83465	4830	0.00028	3	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.2	3.814102564	
Case 13	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	2.259932
Case 14	0.28346	10.00	0.10	2.83465	4830	0.00028	2	100.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.602564103	0.4	1.506410256	
Case 15	0.28346	10.00	0.10	2.83465	4830	0.00028	2	120.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.923076923	0.4	2.307692308	
Case 16	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	0.160513
Case 17	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.28346	10.00	0.10	2.83465	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.5	1.525641026	
Case 18	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.00028	10.00	0.10	0.00283	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 19	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	4.00	0.25	0.11339	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 20	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.28346	4.00	0.25	1.13386	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.5	1.525641026	
Case 21	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.00028	4.00	0.25	0.00113	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 22	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	15.00	0.07	0.42520	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 23	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.28346	15.00	0.07	4.25197	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.5	1.525641026	
Case 24	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.00028	15.00	0.07	0.00425	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	0.038966
Case 25	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 26	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	283.46	4.00	0.25	1133.9	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 27	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	2.83	4.00	0.25	11.3	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 28	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	1.00	1.00	28.3	14.17	10.00	0.10	141.7	0.762820513	0.5	1.525641026	
Case 29	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	283.46	1.00	1.00	283.5	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 30	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	2.83	1.00	1.00	2.8	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	
Case 31	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	10.00	0.10	283.5	14.17	10.00	0.10	141.7	0.762820513	0.3	2.542735043	
Case 32	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	283.46	10.00	0.10	2834.6	14.17	10.00	0.10	141.7	0.762820513	0.3	2.542735043	
Case 33	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	2.83	10.00	0.10	28.3	14.17	10.00	0.10	141.7	0.762820513	0.4	1.907051282	0.108207

Expected Gradient	0.40	variance	2.567619
Expected Factor of Safety	1.91	standard dev	1.602379
	1.910		
V=	0.84		
sigmalnFS	0.73		
E(lnFS)	0.496477419		
beta	0.679308195		
FS critical = 1	1		
ln (FS crit)	0		
Z=	-0.679308195		
Pu=	**0.248471302**		

Critical Gradient = $(\gamma-\gamma_w)/\gamma_w$

Calculated Gradient:

(NO blanket layer) = Max +Y-gradient at toe of landside slope using Seep/W output

(WITH blanket layer) = (Max. ΔHead across blanket layer)/blanket thickness

Table A-5 Winslow Station 29000, Water Surface Elevation 4845 feet

	Levee Fill				Bentonite cutoff		Blanket						Sand Below blanket				SP-CL below sand				Critical Gradient	Calculated Gradient	Factor of Safety	
	kv	kh/kv	$(kh/kv)^{-1}$	kh		kv=kh	thickness	unit weight	kv	kh/kv	$(kh/kv)^{-1}$	kh	kv	kv/kh		kh	kv	kh/kv	$(kh/kv)^{-1}$	kh				
	ft/day			ft/day		ft/day	ft	pcf	ft/day			ft/day	ft/day			ft/day	ft/day			ft/day		(from SEEP/W)		
Case 1	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 2	2.83465	10.00	0.10	28.34646	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 3	0.02835	10.00	0.10	0.28346	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 4	0.28346	4.00	0.25	1.13386	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 5	2.83465	4.00	0.25	11.33858	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 6	0.02835	4.00	0.25	0.11339	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 7	0.28346	15.00	0.07	4.25197	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 8	2.83465	15.00	0.07	42.51969	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 9	0.02835	15.00	0.07	0.42520	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 10	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	5.48E-32
Case 11	0.28346	10.00	0.10	2.83465	4830	0.00028	1	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1.5	0.508547	
Case 12	0.28346	10.00	0.10	2.83465	4830	0.00028	3	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.5	1.525641	
Case 13	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	0.260379
Case 14	0.28346	10.00	0.10	2.83465	4830	0.00028	2	100.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.602564	0.7	0.860806	
Case 15	0.28346	10.00	0.10	2.83465	4830	0.00028	2	120.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.923077	0.7	1.318681	
Case 16	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	0.052412
Case 17	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.28346	10.00	0.10	2.83465	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 18	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.00028	10.00	0.10	0.00283	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 19	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	4.00	0.25	0.11339	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 20	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.28346	4.00	0.25	1.13386	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 21	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.00028	4.00	0.25	0.00113	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 22	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	15.00	0.07	0.42520	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 23	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.28346	15.00	0.07	4.25197	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 24	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.00028	15.00	0.07	0.00425	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	5.63E-32
Case 25	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 26	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	283.46	4.00	0.25	1133.9	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 27	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	2.83	4.00	0.25	11.3	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 28	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	1.00	1.00	28.3	14.17	10.00	0.10	141.7	0.762821	0.8	0.953526	
Case 29	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	283.46	1.00	1.00	283.5	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 30	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	2.83	1.00	1.00	2.8	14.17	10.00	0.10	141.7	0.762821	0.8	0.953526	
Case 31	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	10.00	0.10	283.5	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 32	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	283.46	10.00	0.10	2834.6	14.17	10.00	0.10	141.7	0.762821	0.6	1.271368	
Case 33	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	2.83	10.00	0.10	28.3	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	0.008648

Expected Gradient	0.7	variance	0.32144
Expected Factor of Safety	1.09	std dev	0.566956
	1.090		
V=	0.52		
sigmalnFS	0.49		
E(lnFS)	-0.030227018		
beta	-0.061758633		
FS critical =	1		
ln (FS crit)	0		
Z=	0.061758633		
Pu=	**0.524622477**		

Critical Gradient = $(\gamma-\gamma_w)/\gamma_w$

Calculated Gradient:

(NO blanket layer) = Max +Y-gradient at toe of landside slope using Seep/W output

(WITH blanket layer) = (Max. ΔHead across blanket layer)/blanket thickness

Table A-6 Winslow Station 29000, Water Surface Elevation 4847 feet

	Levee Fill				Bentonite cutoff		Blanket						Sand Below blanket				SP-CL below sand				Critical Gradi	Calculatec	Factor of Safety	
	kv	kh/kv	$(kh/kv)^{-1}$	kh		kv=kh	thickness	ınit weigh	kv	kh/kv	$(kh/kv)^{-1}$	kh	kv	kv/kh		kh	kv	kh/kv	$(kh/kv)^{-1}$	kh				
	cm/sec			cm/sec		cm/sec	ft	pcf	cm/sec			cm/sec	cm/sec			cm/sec	cm/sec			cm/sec		(from SEEP/W)		
Case 1	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 2	2.83465	10.00	0.10	28.34646	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 3	0.02835	10.00	0.10	0.28346	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 4	0.28346	4.00	0.25	1.13386	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 5	2.83465	4.00	0.25	11.33858	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 6	0.02835	4.00	0.25	0.11339	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 7	0.28346	15.00	0.07	4.25197	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 8	2.83465	15.00	0.07	42.51969	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 9	0.02835	15.00	0.07	0.42520	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 10	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	0
Case 11	0.28346	10.00	0.10	2.83465	4830	0.00028	1	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	2	0.38141	
Case 12	0.28346	10.00	0.10	2.83465	4830	0.00028	3	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 13	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	0.125681
Case 14	0.28346	10.00	0.10	2.83465	4830	0.00028	2	100.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.602564	1	0.602564	
Case 15	0.28346	10.00	0.10	2.83465	4830	0.00028	2	120.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.923077	1	0.923077	
Case 16	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	0.025682
Case 17	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.28346	10.00	0.10	2.83465	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 18	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.00028	10.00	0.10	0.00283	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 19	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	4.00	0.25	0.11339	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 20	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.28346	4.00	0.25	1.13386	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 21	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.00028	4.00	0.25	0.00113	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1.1	0.693473	
Case 22	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	15.00	0.07	0.42520	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 23	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.28346	15.00	0.07	4.25197	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	0.7	1.089744	
Case 24	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.00028	15.00	0.07	0.00425	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1.1	0.693473	0.034517
Case 25	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	4.00	0.25	113.4	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 26	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	283.46	4.00	0.25	1133.9	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 27	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	2.83	4.00	0.25	11.3	14.17	10.00	0.10	141.7	0.762821	1.1	0.693473	
Case 28	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	1.00	1.00	28.3	14.17	10.00	0.10	141.7	0.762821	1.1	0.693473	
Case 29	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	283.46	1.00	1.00	283.5	14.17	10.00	0.10	141.7	0.762821	1.1	0.693473	
Case 30	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	2.83	1.00	1.00	2.8	14.17	10.00	0.10	141.7	0.762821	2.8	0.272436	
Case 31	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	28.35	10.00	0.10	283.5	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 32	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	283.46	10.00	0.10	2834.6	14.17	10.00	0.10	141.7	0.762821	1	0.762821	
Case 33	0.28346	10.00	0.10	2.83465	4830	0.00028	2	110.00	0.02835	10.00	0.10	0.28346	2.83	10.00	0.10	28.3	14.17	10.00	0.10	141.7	0.762821	1	0.762821	0.025088
																					Expected Gradient	1	variance	0.210968
																					Expected Factor of Safety	0.76	std dev	0.459313
																						0.760		
																					V=	0.60		
																					sigmalnFS	0.56		
																					E(lnFS)	-0.502		
																					beta	-0.90255		
																					FS critical =	1		
																					ln (FS crit)	0		
																					Z=	0.902547		
																					Pu=	**0.816617**		

Critical Gradient = $(\gamma-\gamma_w)/\gamma_w$

Calculated Gradient:

(NO blanket layer) = Max +Y-gradient at toe of landside slope using Seep/W output

(WITH blanket layer) = (Max. ΔHead across blanket layer)/blanket thickness

Table A-7 Winslow Station 29000, Water Surface Elevation 4849 feet

Example Geo-Studio Seep/W Calculation Input Report

Station 37000

Steady-State Seepage

File Information

Created By: Brown, Stephen
Revision Number: 488
Last Edited By: Brown, Stephen
Date: 3/21/2012
Time: 7:18:28 AM
File Name: 37000 100-yr EL 4852_39.gsz
Directory: C:\Documents and Settings\L1COASLB\My Documents\Projects\Winslow Levees\Winslow GeoStudio\37000\

Project Settings

Length(L) Units: feet
Time(t) Units: Days
Force(F) Units: lbf
Pressure(p) Units: psf
Mass(M) Units: lbs
Mass Flux Units: lbs/days
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Steady-State Seepage

Kind: SEEP/W
Method: Steady-State
Settings
 Include Air Flow: No
Control
 Apply Runoff: Yes
Convergence
 Convergence Type: Gauss Point K
 Convergence Settings
  Maximum Number of Iterations: 500
  Tolerance: 0.01
  Maximum Change in K: 0.1
  Rate of Change in K: 1.02
  Minimum Change in K: 0.0001
 Equation Solver: Parallel Direct
 Potential Seepage Max # of Reviews: 10
Time
 Starting Time: 0 days
 Duration: 0 days
 Ending Time: 0 days

Materials

Levee Fill

Model: Saturated Only
Hydraulic
 K-Sat: 0.002835 ft/days
 Volumetric Water Content: 0 ft^3/ft^3
 Mv: 0 /psf
 K-Ratio: 0.1
 K-Direction: 0 °

Sand (SP/SM)

Model: Saturated Only
Hydraulic
 K-Sat: 28.35 ft/days
 Volumetric Water Content: 0 ft^3/ft^3
 Mv: 0 /psf
 K-Ratio: 0.2
 K-Direction: 0 °

Clay (CL/CH)

Model: Saturated Only
Hydraulic
 K-Sat: 0.002835 ft/days
 Volumetric Water Content: 0 ft^3/ft^3
 Mv: 0 /psf
 K-Ratio: 0.1
 K-Direction: 0 °

Bentonite Cutoff

Model: Saturated Only
Hydraulic
 K-Sat: 0.0002835 ft/days
 Volumetric Water Content: 0 ft^3/ft^3
 Mv: 0 /psf
 K-Ratio: 1
 K-Direction: 0 °

Boundary Conditions

Potential Seepage Face

Review: true
Type: Total Flux (Q) 0

Reservoir Head

Type: Head (H) 852.39

GWT

Type: Head (H) 837

Flux Sections

Flux Section 1

Coordinates

Coordinate: (248.55284, 479.48026) ft

Coordinate: (248.8495, 620.98784) ft

Regions

	Material	Points	Area (ft²)
Region 1	Bentonite Cutoff	5,21,19,9,3,4,10,20,22,6	46
Region 2	Sand (SP/SM)	28,27,23,9,19	297
Region 3	Levee Fill	23,7,1,2,8,10,4,3,9	681.2
Region 4	Sand (SP/SM)	10,8,24,25,20	840
Region 5	Sand (SP/SM)	29,28,19,21,5,6,22,20,25,26,12,14,16	6725
Region 6	Sand (SP/SM)	28,13,11,27	401
Region 7	Sand (SP/SM)	15,13,28,29	4059
Region 8	Clay (CL/CH)	17,15,29,16,18	6000

Lines

	Start Point	End Point	Hydraulic Boundary
Line 1	10	4	
Line 2	4	3	
Line 3	3	9	
Line 4	5	21	
Line 5	21	19	
Line 6	19	9	
Line 7	10	20	
Line 8	20	22	
Line 9	22	6	
Line 10	6	5	
Line 11	8	10	
Line 12	9	23	
Line 13	28	27	
Line 14	27	23	Potential Seepage Face
Line 15	19	28	
Line 16	29	28	
Line 17	16	29	
Line 18	23	7	Potential Seepage Face
Line 19	7	1	Potential Seepage Face
Line 20	1	2	
Line 21	2	8	Reservoir Head
Line 22	8	24	Reservoir Head
Line 23	24	25	Reservoir Head
Line 24	25	20	
Line 25	25	26	Reservoir Head
Line 26	26	12	Reservoir Head
Line 27	12	14	Reservoir Head
Line 28	14	16	Reservoir Head
Line 29	28	13	

Line 30	13	11	GWT
Line 31	11	27	Potential Seepage Face
Line 32	15	13	GWT
Line 33	29	15	
Line 34	17	15	GWT
Line 35	16	18	Reservoir Head
Line 36	18	17	

Points

	X (ft)	Y (ft)
Point 1	239	855.2
Point 2	261	855.2
Point 3	249	853
Point 4	251	853
Point 5	249	830
Point 6	251	830
Point 7	209	842
Point 8	279	846
Point 9	249	840
Point 10	251	840
Point 11	-100	840
Point 12	500	838
Point 13	-100	838
Point 14	500	835
Point 15	-100	820
Point 16	500	820
Point 17	-100	810
Point 18	500	810
Point 19	249	838
Point 20	251	838
Point 21	249	833
Point 22	251	833
Point 23	201	840
Point 24	389	842
Point 25	409	838
Point 26	450	838
Point 27	100	840
Point 28	101	838
Point 29	150	820

Appendix B: Slope Stability Calculation Summaries

The following table summarizes the calculations of slope stability. Each cross section was evaluated at different water levels using randomly generated soil properties (generated in accordance with defined distribution) 2000 times to estimate a distribution of factors of safety, and determine the probability that the factor of safety of 1.0 is reached for the section. Slip surfaces were defined such that the slip surfaces had to involve a majority of the levee. Less than 10-feet of crest outside of the slip circle with a Factor of Safety less than 1.0 was defined as unsatisfactory performance. The distribution of soil properties is discussed in the main portion of the text.

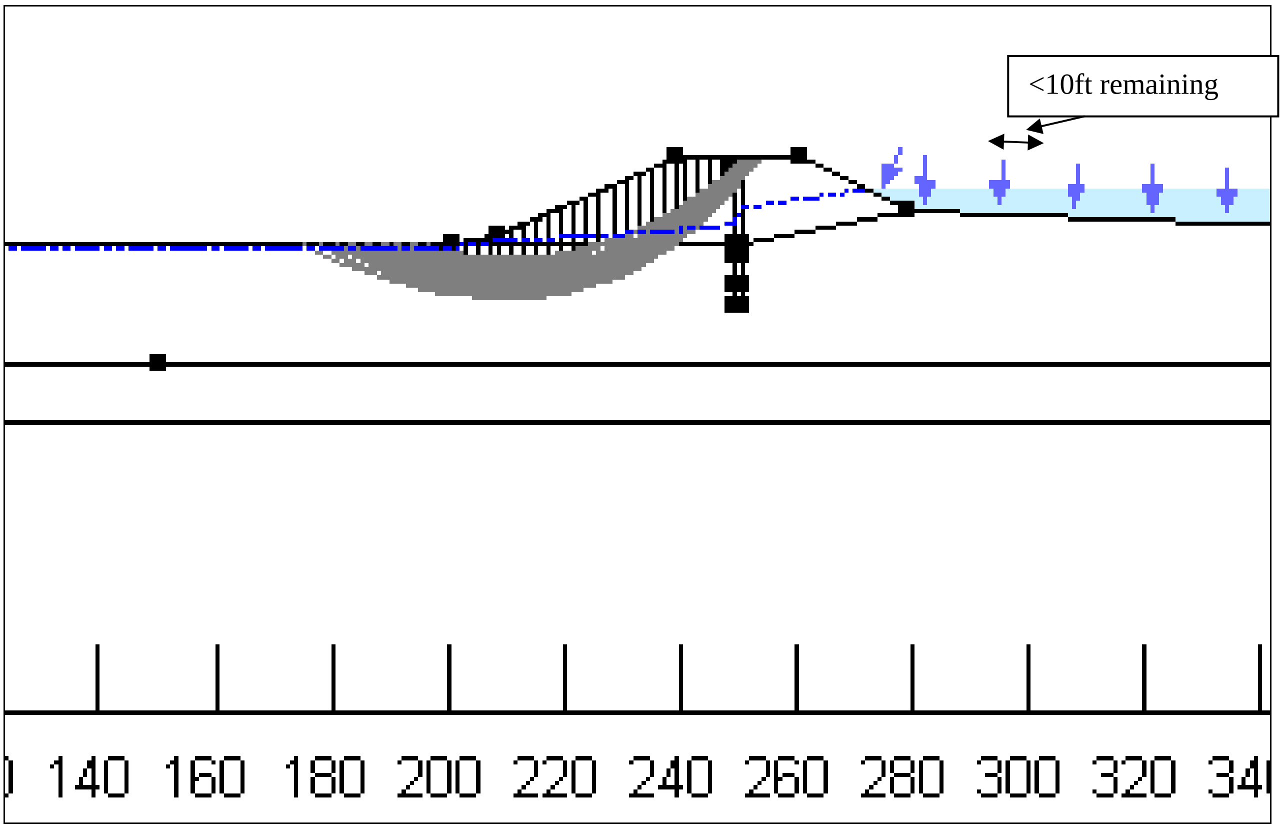


Figure B-1 Illustration of Potential Slip Surfaces that are Considered Unsatisfactory Performance

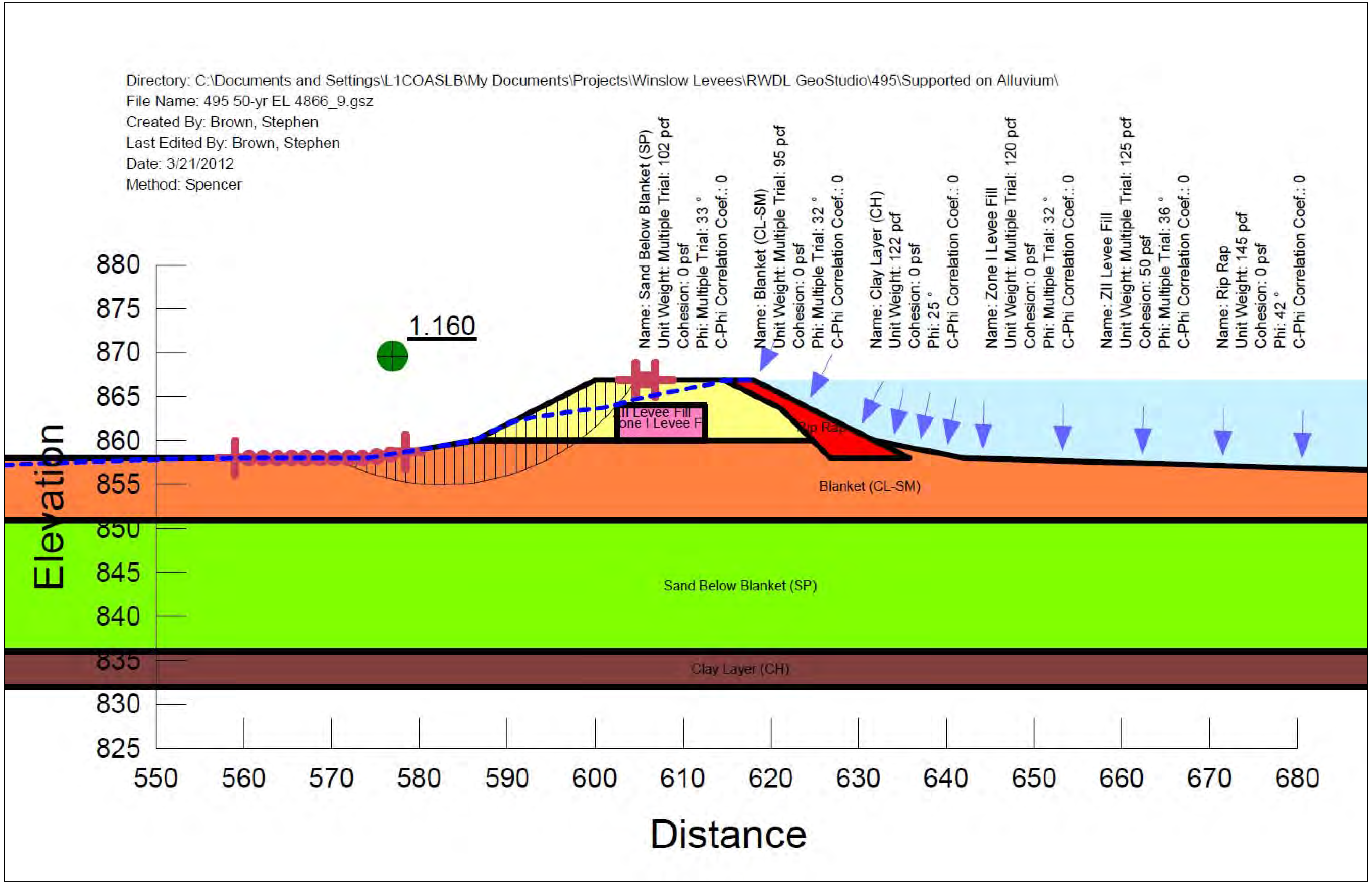
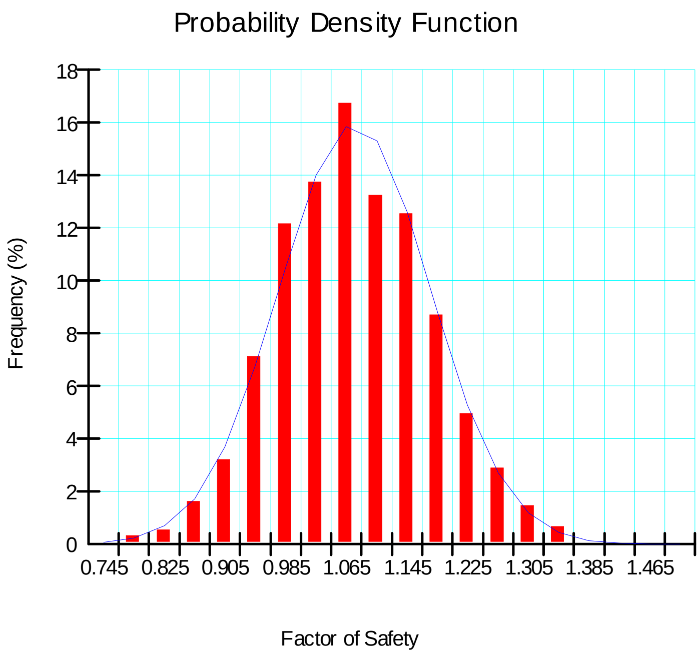
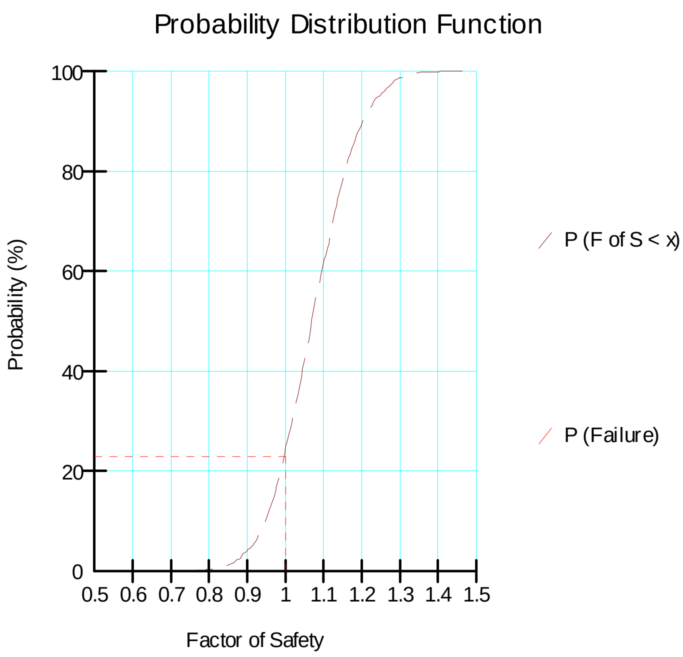
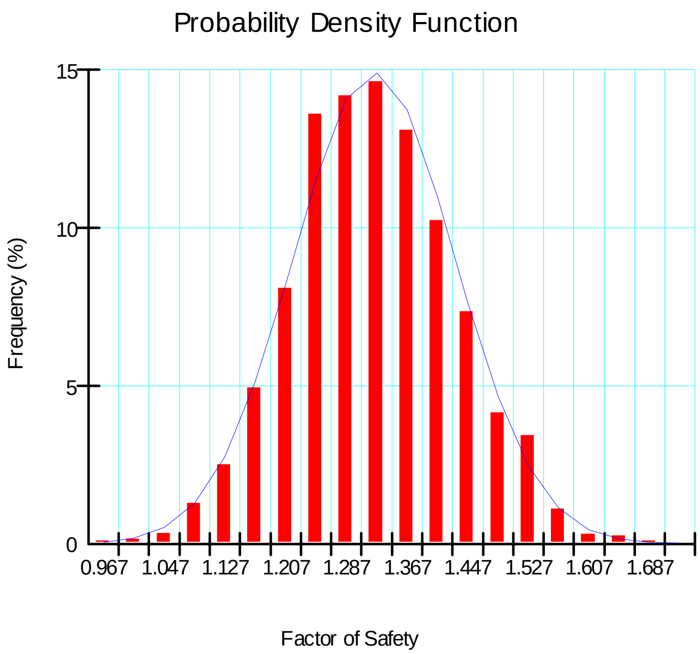


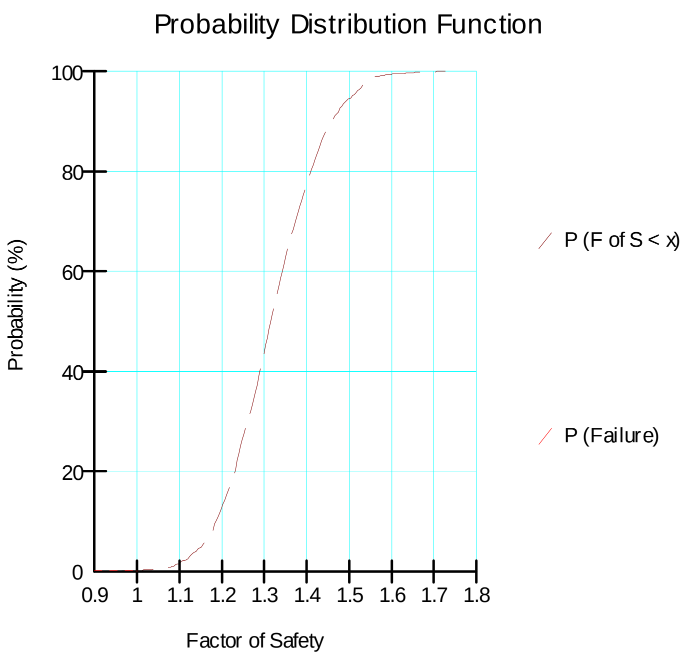
Figure B-2 RWDL 495 Slope Stability Calculation Output Example (units in feet)



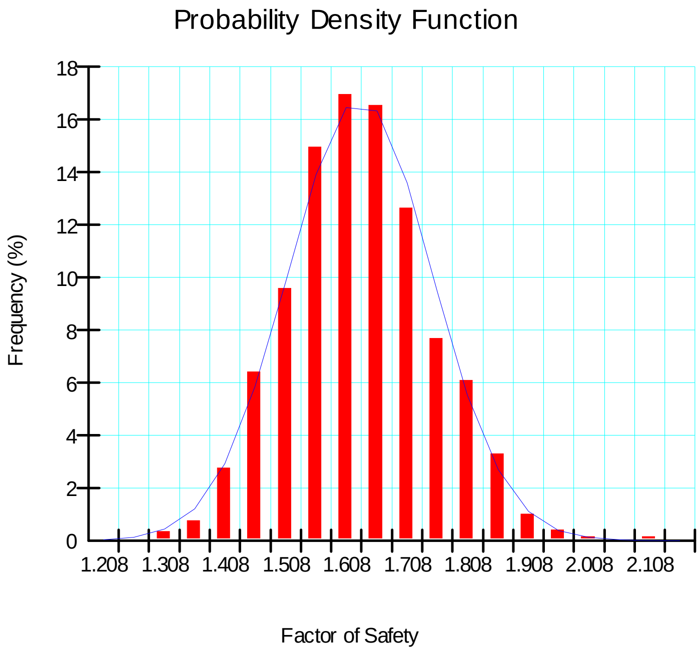


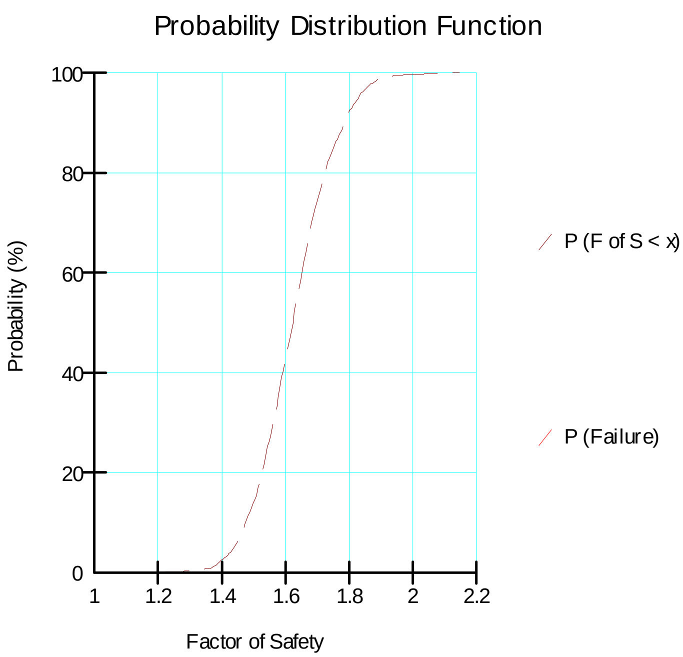
Graphs B-1 Factor of Safety Distribution RWDL 495 Alluvium Foundation, Water Surface Elevation 4867 feet



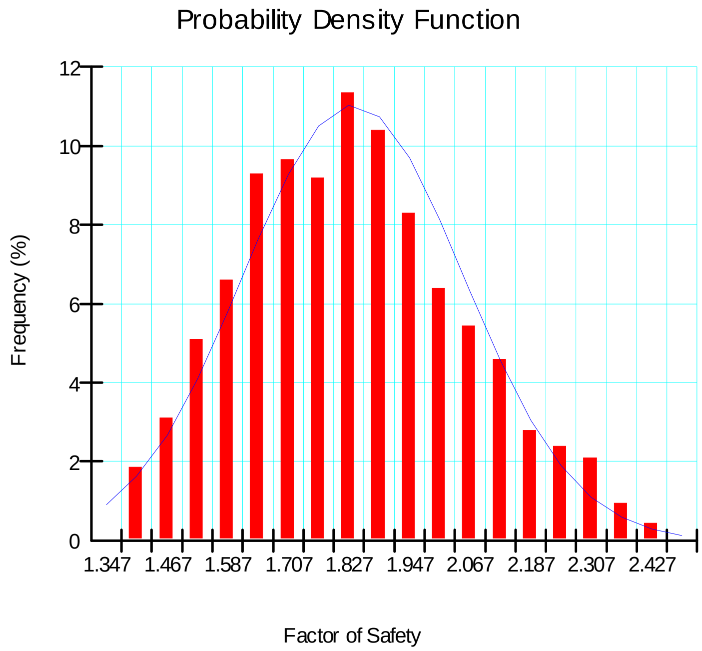


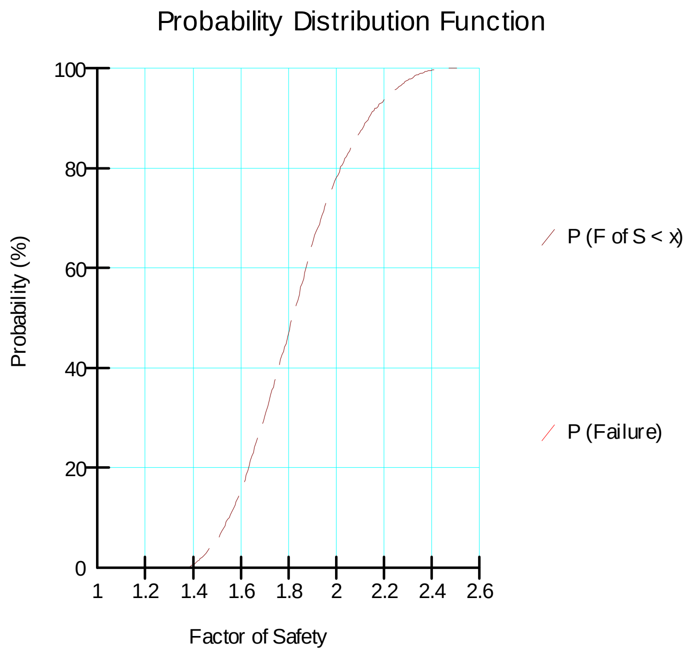
Graphs B-2 Factor of Safety Distribution RWDL 495 Alluvium Foundation, Water Surface Elevation 4864.5feet



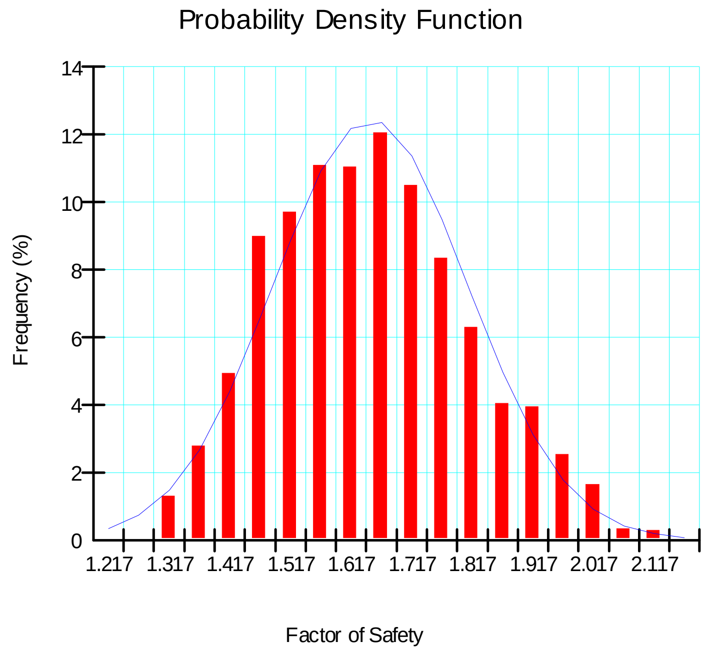


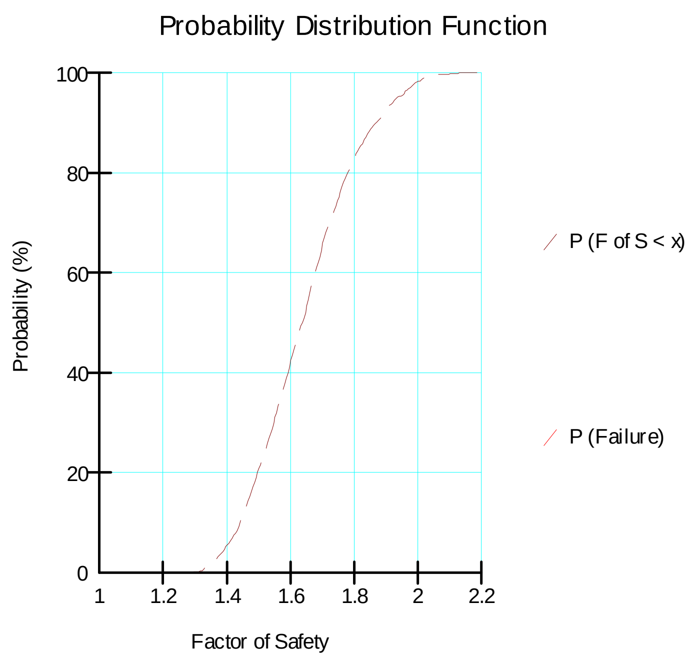
Graphs B-3 Factor of Safety Distribution RWDL 495 Alluvium Foundation, Water Surface Elevation 4862 feet



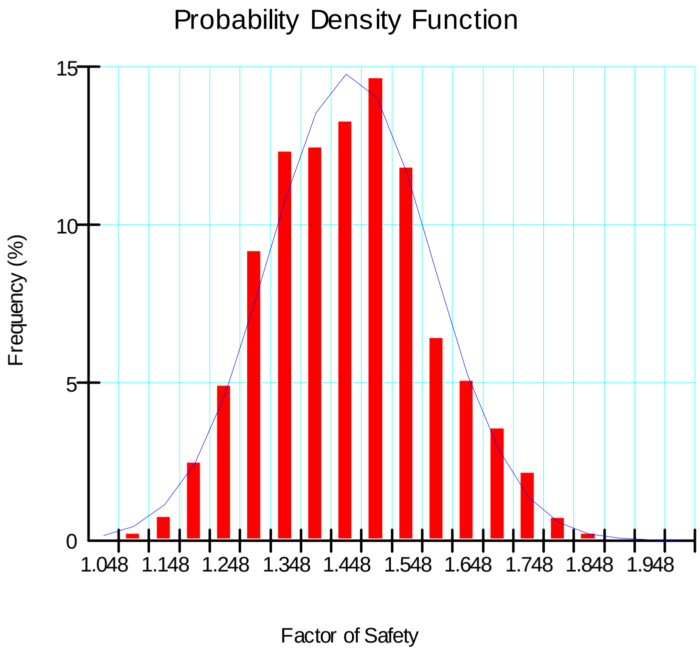


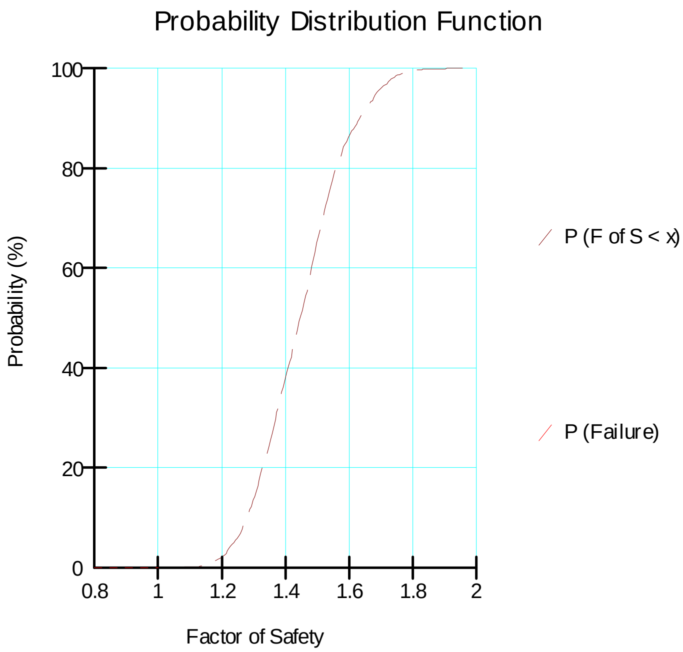
Graphs B-4 Factor of Safety Distribution RWDL 495 Sandstone Foundation, Water Surface Elevation 4862 feet





Graphs B-5 Factor of Safety Distribution RWDL 495 Sandstone Foundation, Water Surface Elevation 4864.6 feet





Graphs B-6 Factor of Safety Distribution RWDL 495 Sandstone Foundation, Water Surface Elevation 4867 feet

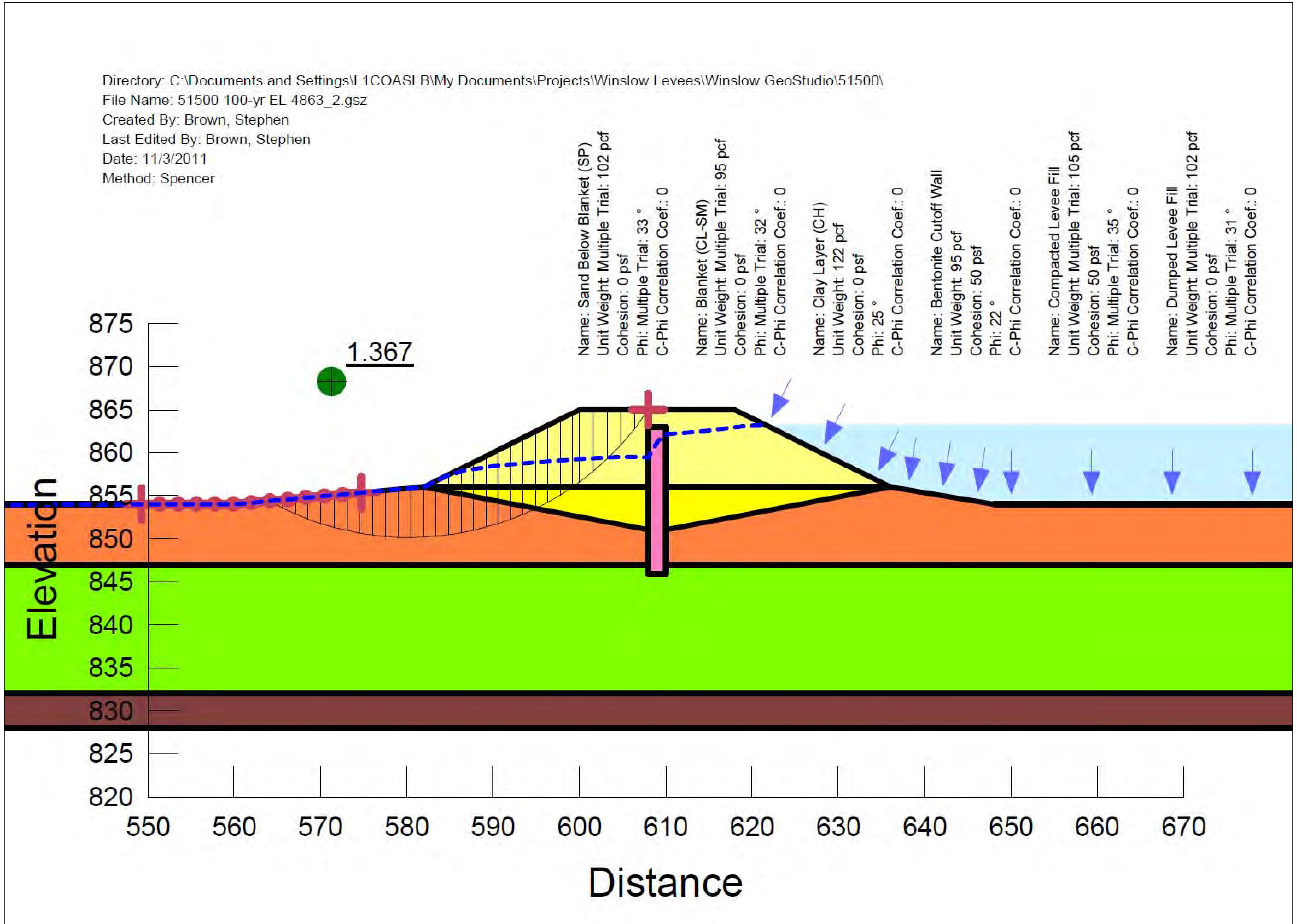
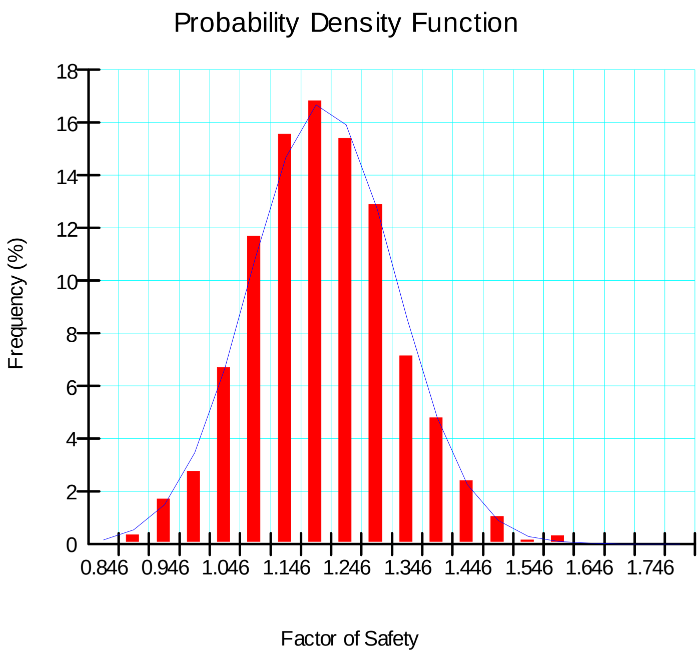
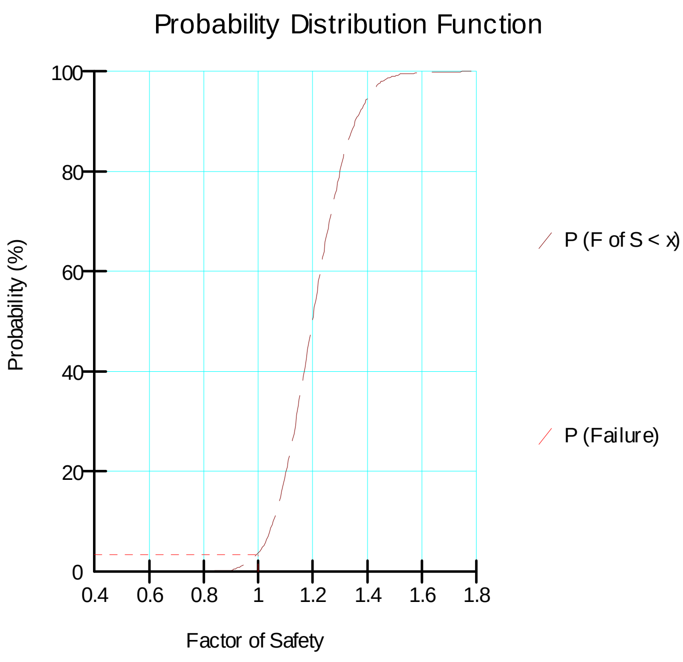
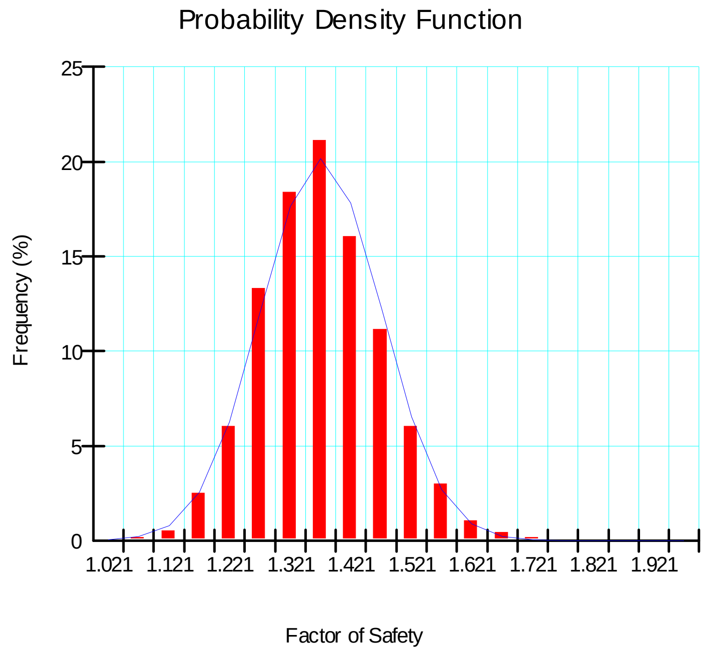


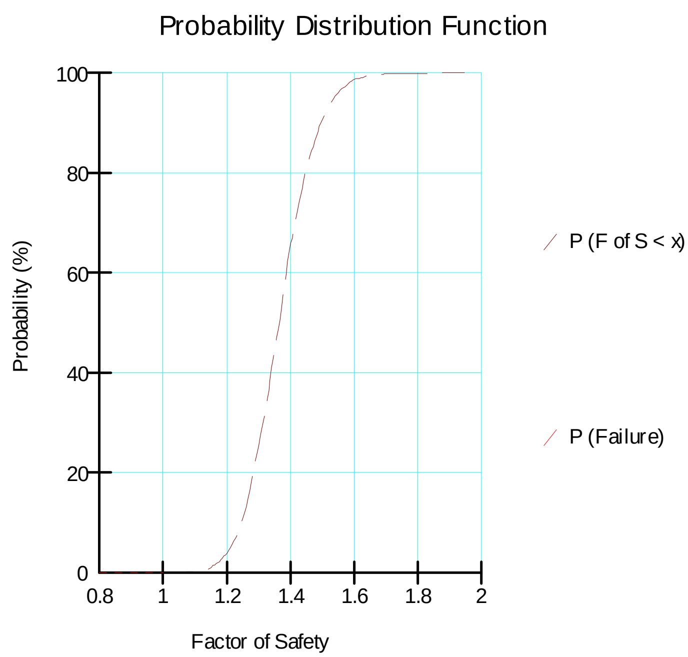
Figure B-3 Winslow 51500 Slope Stability Calculation Output Example (units in feet)



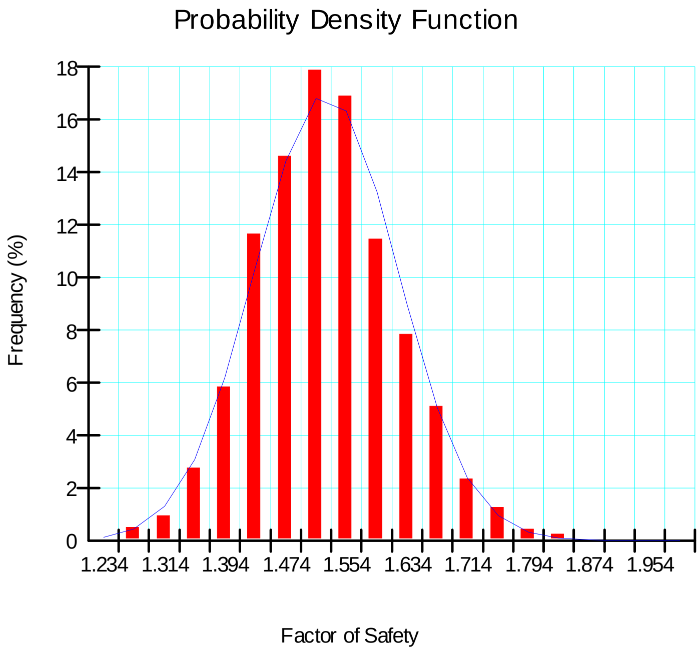


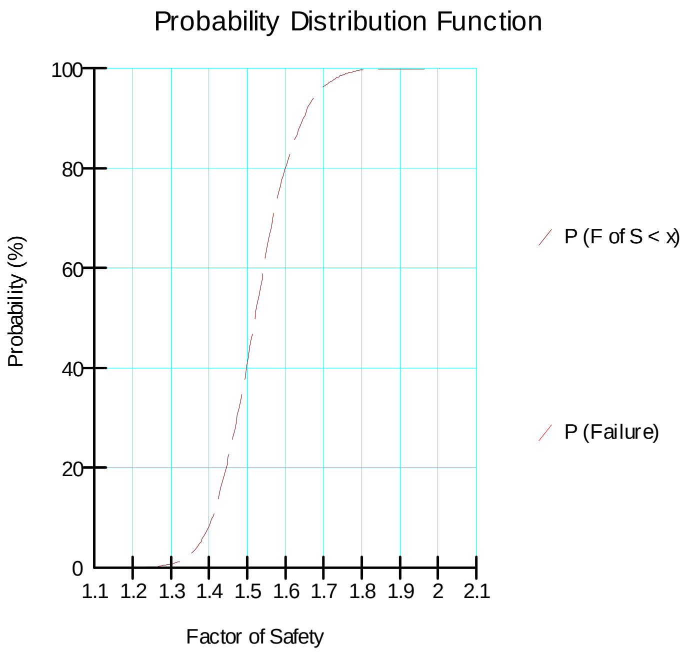
Graphs B-7 Factor of Safety Distribution Winslow Station 51500, Water Surface Elevation 4865 feet



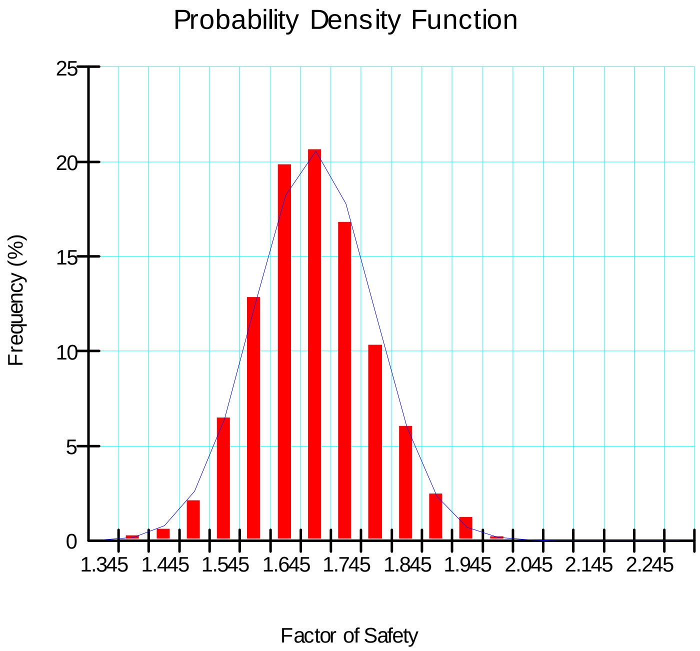


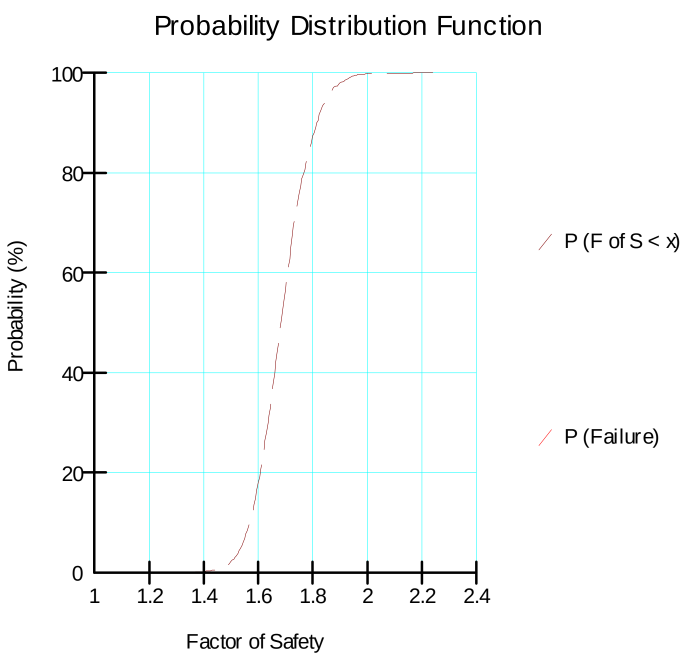
Graphs B-8 Factor of Safety Distribution Winslow Station 51500, Water Surface Elevation 4863 feet



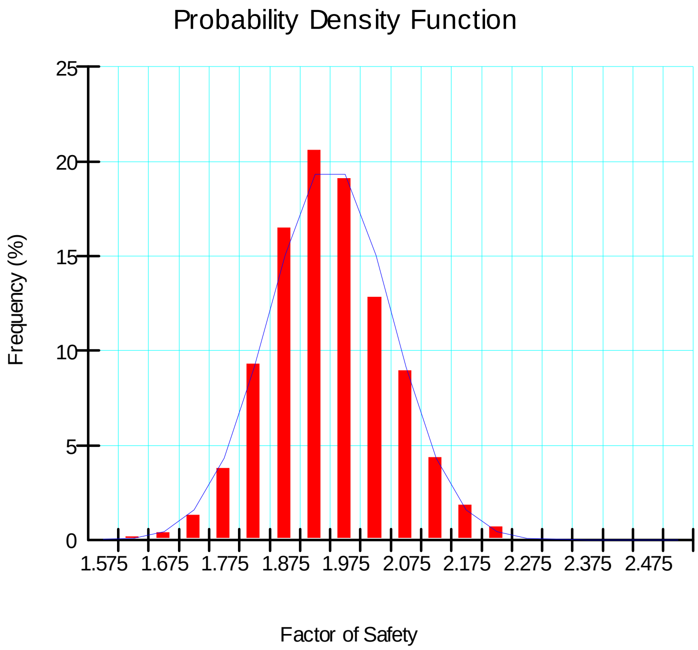


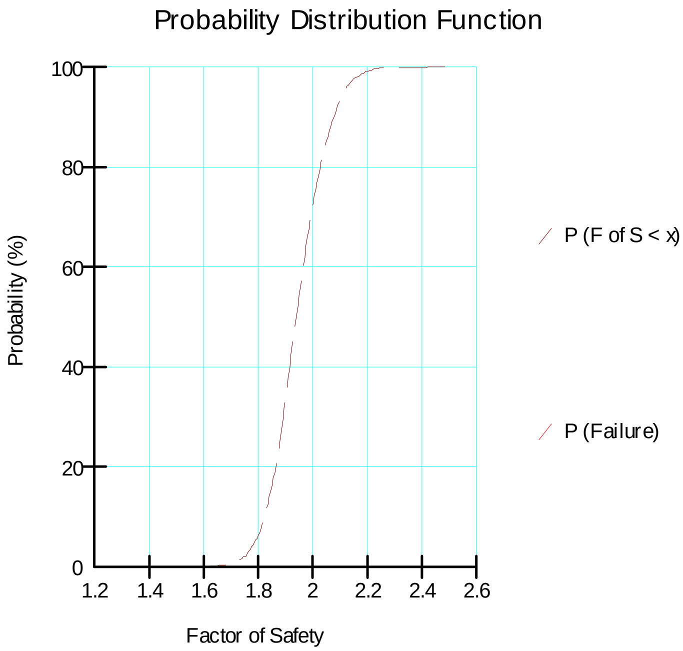
Graphs B-9 Factor of Safety Distribution Winslow Station 51500, Water Surface Elevation 4861 feet





Graphs B-10 Factor of Safety Distribution Winslow Station 51500, Water Surface Elevation 4859.5 feet





Graphs B-11 Factor of Safety Distribution Winslow Station 51500, Water Surface Elevation 4857 feet.

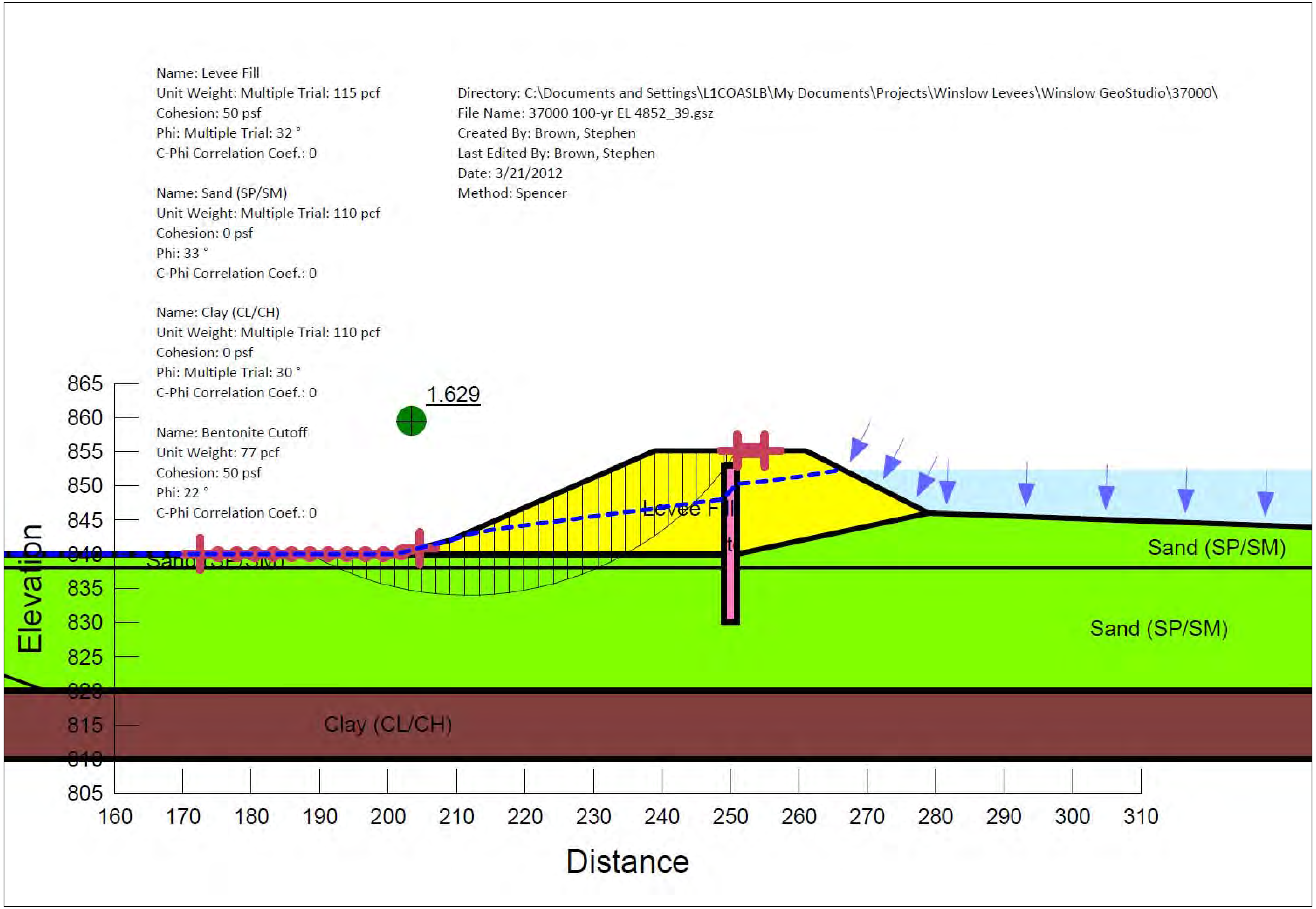
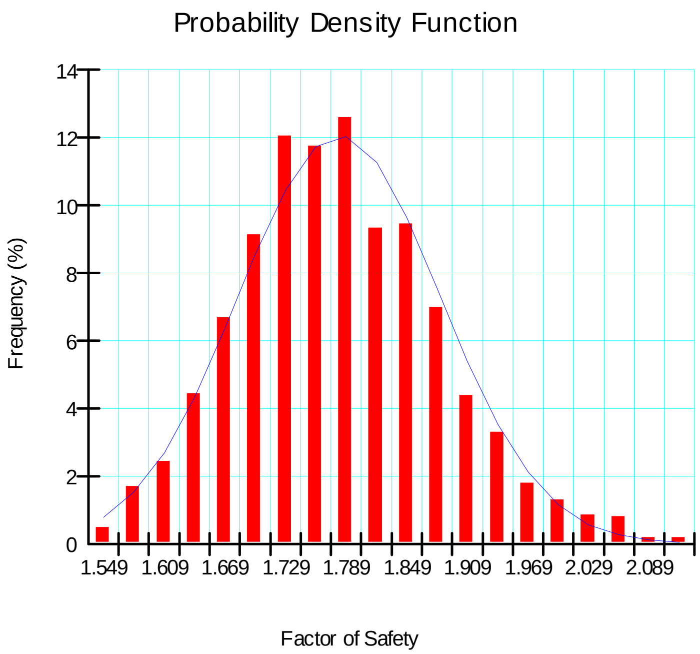
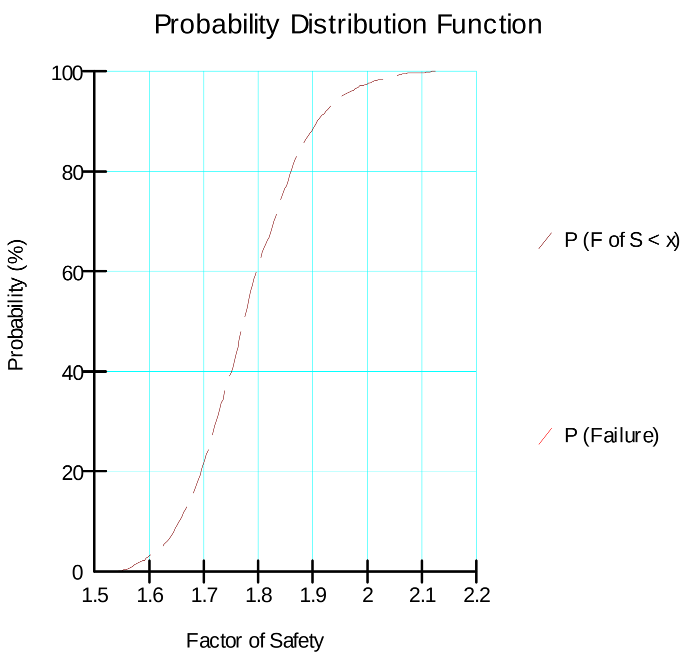
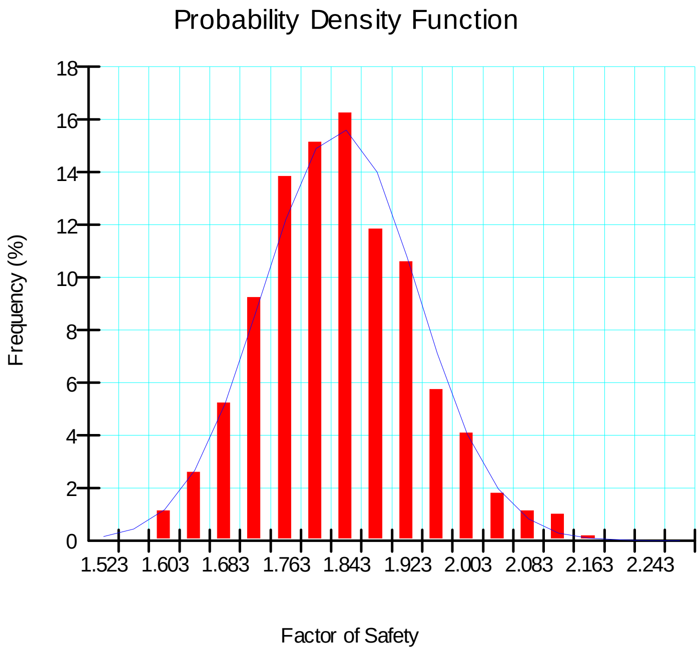


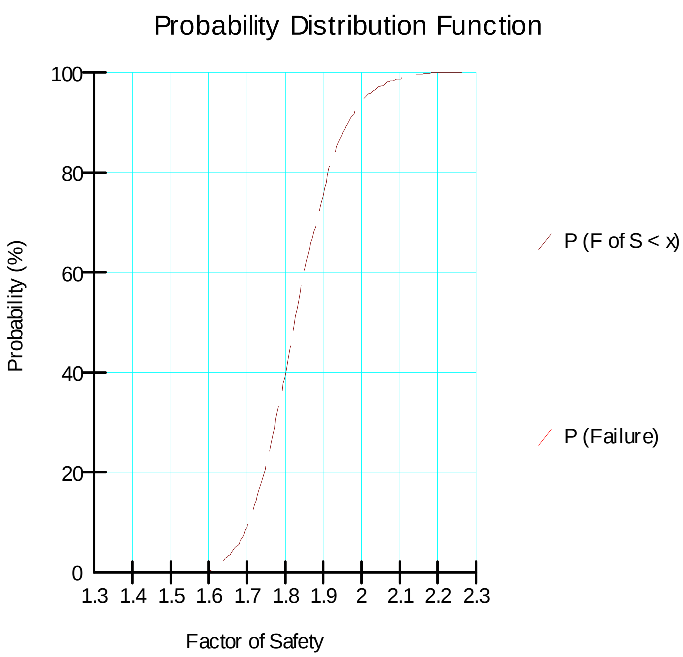
Figure B-4 Winslow 37000 Slope Stability Calculation Output Example (units in feet)



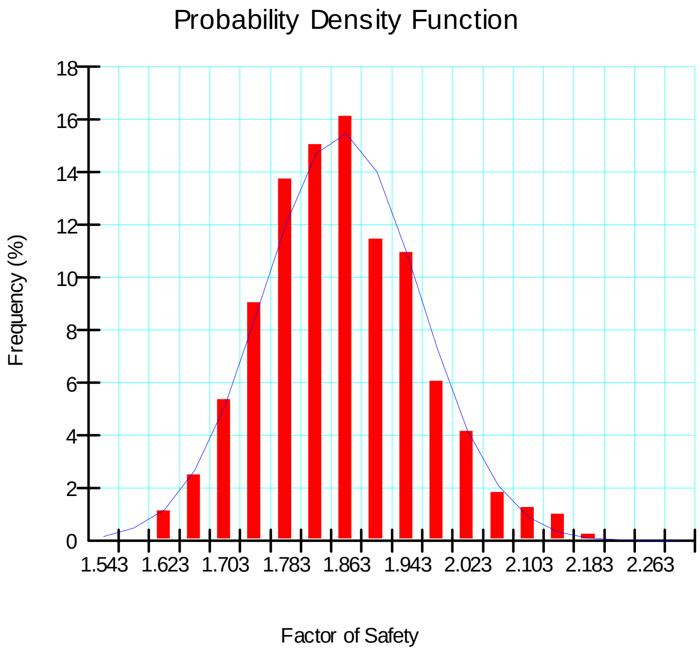


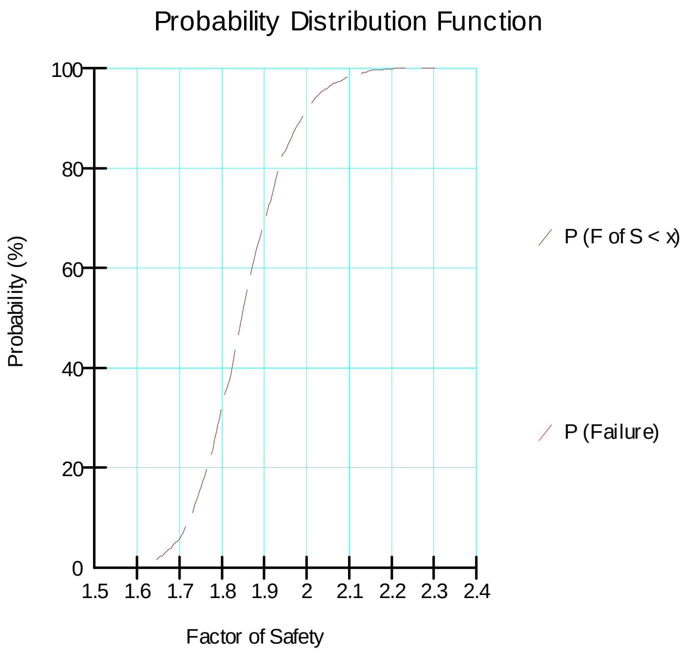
Graphs B-12 Factor of Safety Distribution Winslow Station 37000, Water Surface Elevation 4855 feet



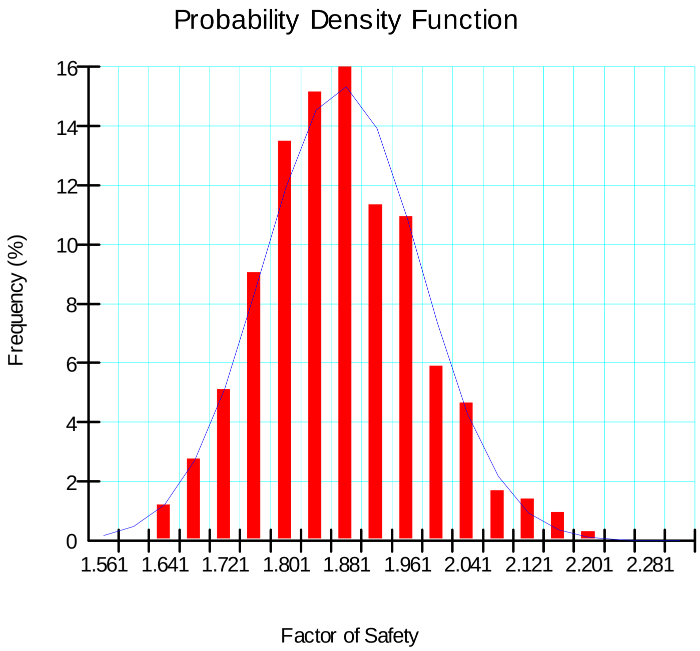


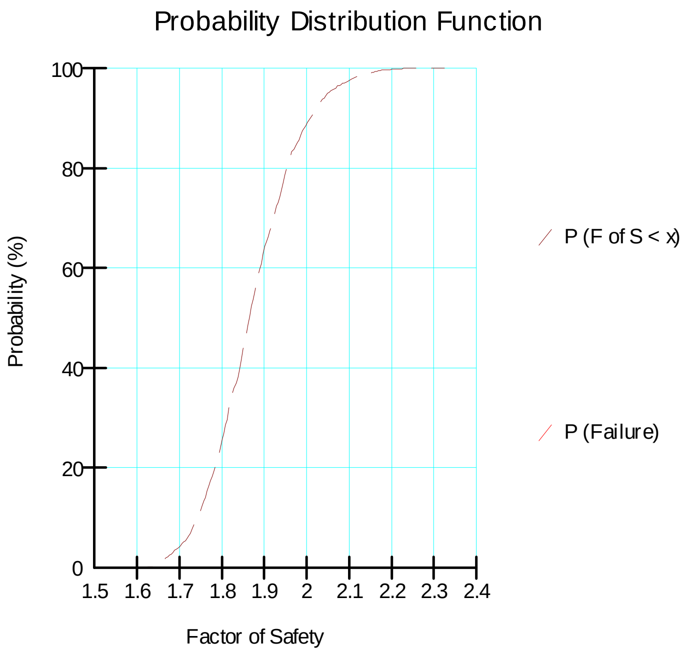
Graphs B-13 Factor of Safety Distribution Winslow Station 37000, Water Surface Elevation 4852 feet



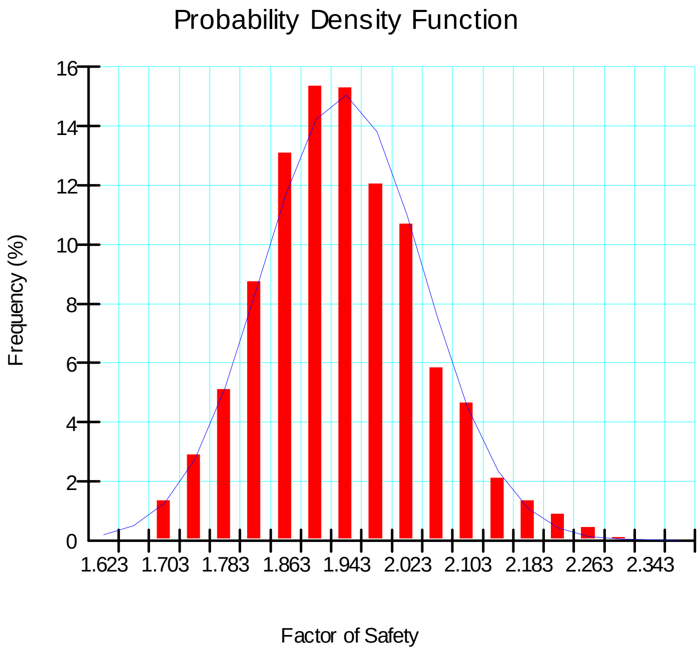


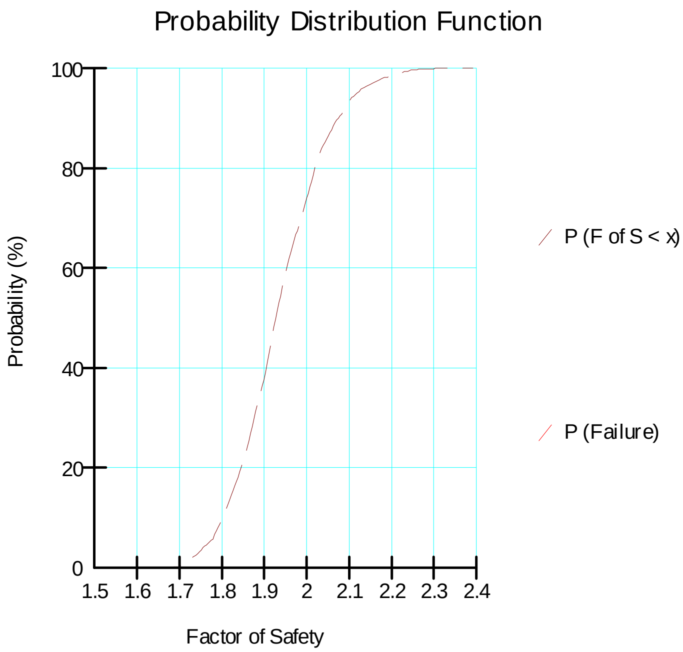
Graphs B-14 Factor of Safety Distribution Winslow Station 37000, Water Surface Elevation 4851 feet



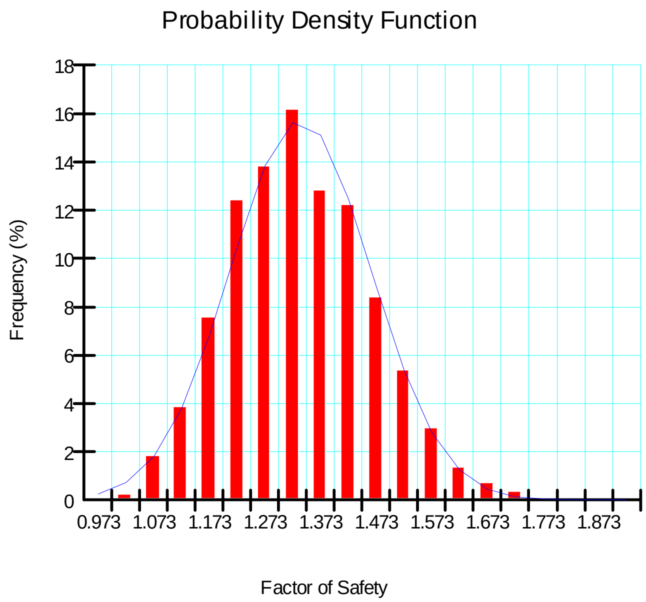


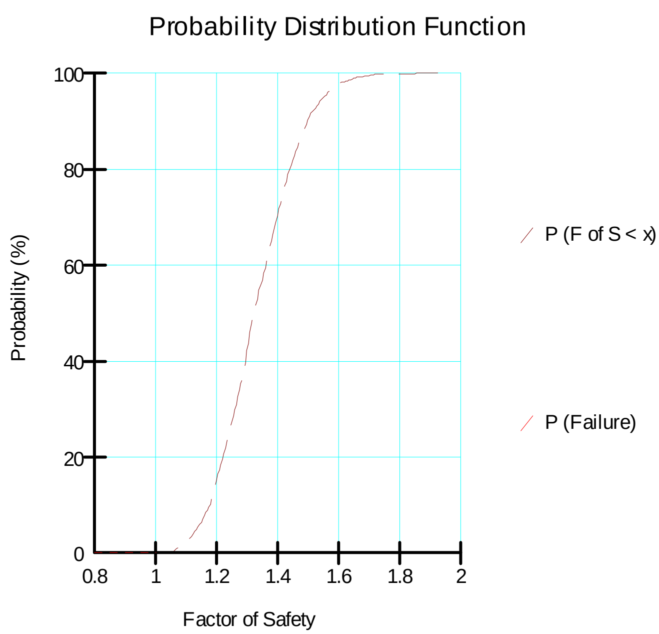
Graphs B-15 Factor of Safety Distribution Winslow Station 37000, Water Surface Elevation 4850 feet





Graphs B-17 Factor of Safety Distribution Winslow Station 37000, Water Surface Elevation 4845 feet





Graphs B-18 Factor of Safety Distribution Winslow Station 37000, Water Surface Elevation 4849 feet

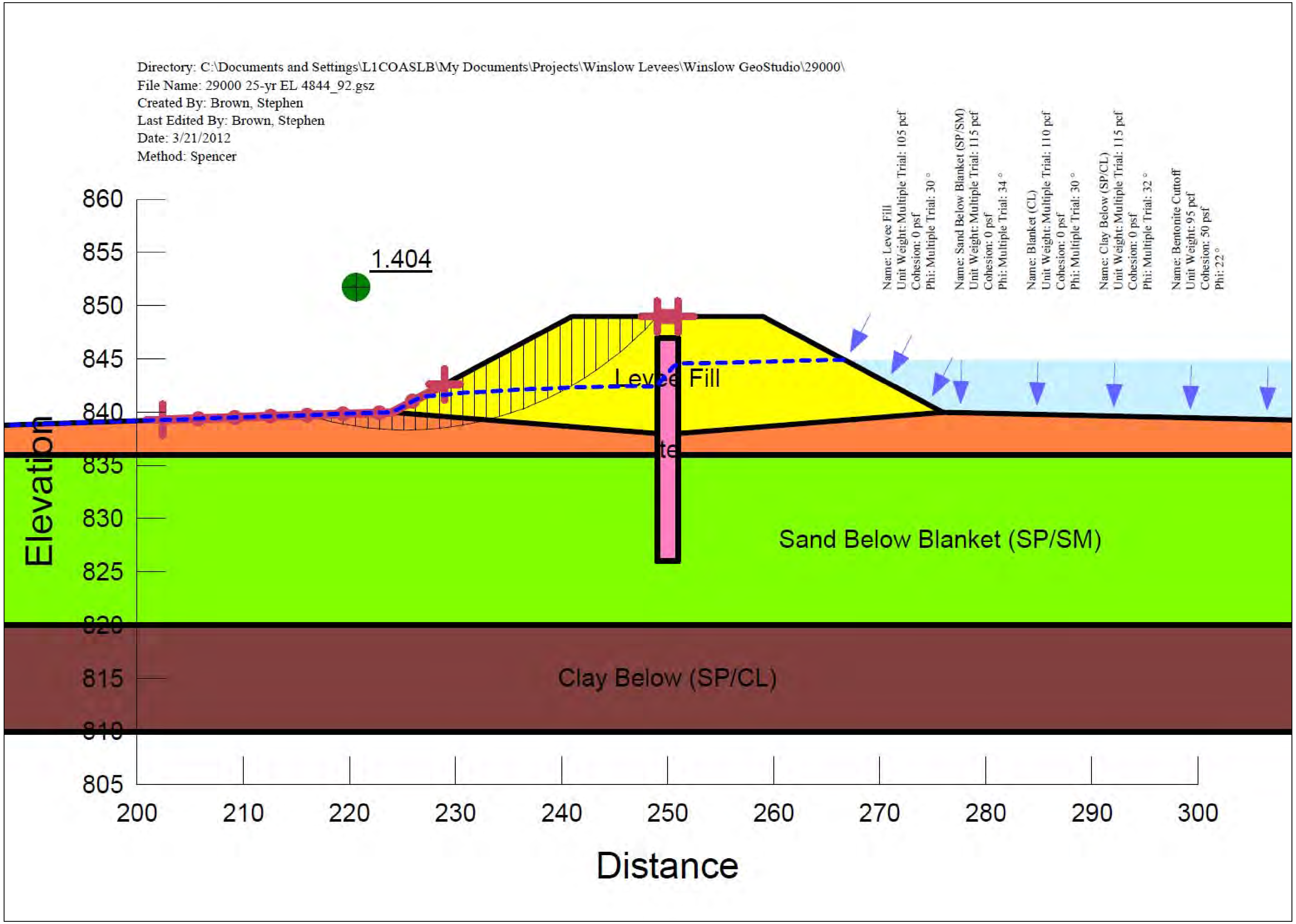
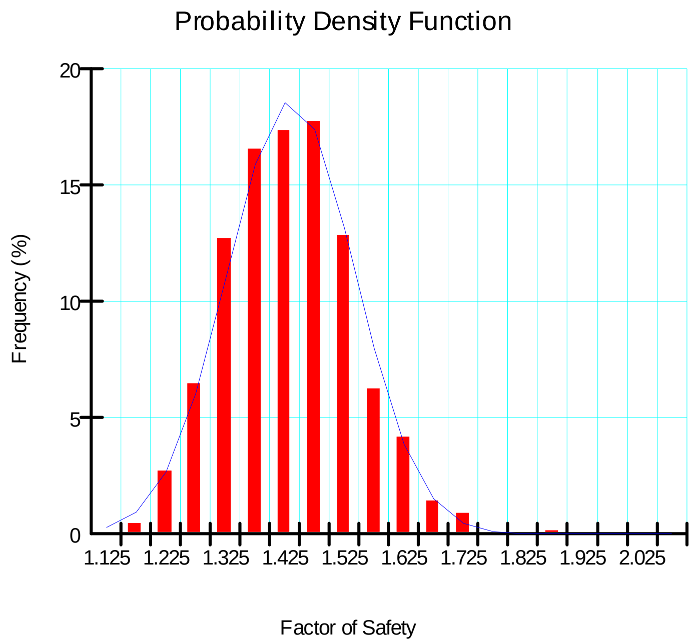
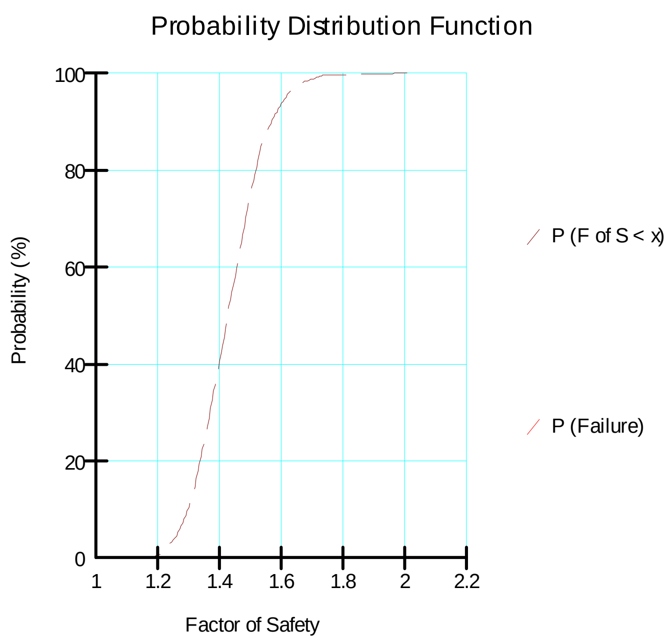


Figure B-4 Winslow 29000 Slope Stability Calculation Output Example (units in feet)





Graphs B-19 Factor of Safety Distribution Winslow Station 29000 Water Surface Elevation 4847 feet

Example Geo-Studio Slope /W Calculation Input Report

Station 29000

Slope Stability

File Information

Created By: Brown, Stephen
Revision Number: 450
Last Edited By: Brown, Stephen
Date: 3/21/2012
Time: 7:42:03 AM
File Name: 29000 25-yr EL 4844_92.gsz
Directory: C:\Documents and Settings\L1COASLB\My Documents\Projects\Winslow Levees\Winslow GeoStudio\29000\

Project Settings

Length(L) Units: feet
Time(t) Units: Days
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D

Analysis Settings

Slope Stability

Kind: SLOPE/W
Parent: Steady-State Seepage
Method: Spencer
Settings
    PWP Conditions Source: Parent Analysis
Slip Surface
    Direction of movement: Right to Left
    Use Passive Mode: No
    Slip Surface Option: Entry and Exit
    Critical slip surfaces saved: 1
    Optimize Critical Slip Surface Location: No
    Tension Crack
        Tension Crack Option: (none)
FOS Distribution
    FOS Calculation Option: Constant
Advanced
    Number of Slices: 30
    Optimization Tolerance: 0.01
    Minimum Slip Surface Depth: 0.1 ft
    Optimization Maximum Iterations: 2000
    Optimization Convergence Tolerance: 1e-007
    Starting Optimization Points: 8
    Ending Optimization Points: 16

Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Levee Fill

Model: Mohr-Coulomb
Unit Weight: Multiple Trial: 105 pcf
Constant Value: 105
Probabilistic: Normal(Mean=105,SD=10,Min=85,Max=145)
Unit Wt. Above Water Table: 135 pcf
Cohesion: 0 psf
Phi: Multiple Trial: 30 °
Constant Value: 30
Probabilistic: Normal(Mean=30,SD=2,Min=28)
Phi-B: 0 °

Sand Below Blanket (SP/SM)

Model: Mohr-Coulomb
Unit Weight: Multiple Trial: 115 pcf
Constant Value: 115
Probabilistic: Normal(Mean=115,SD=10,Min=90,Max=145)
Unit Wt. Above Water Table: 135 pcf
Cohesion: 0 psf
Phi: Multiple Trial: 34 °
Constant Value: 34
Probabilistic: Normal(Mean=34,SD=3,Min=28,Max=45)
Phi-B: 0 °

Blanket (CL)

Model: Mohr-Coulomb
Unit Weight: Multiple Trial: 110 pcf
Constant Value: 110
Probabilistic: Normal(Mean=110,SD=10,Min=80,Max=145)
Unit Wt. Above Water Table: 119 pcf
Cohesion: 0 psf
Phi: Multiple Trial: 30 °
Constant Value: 30
Probabilistic: Normal(Mean=30,SD=3,Min=22)
Phi-B: 0 °

Clay Below (SP/CL)

Model: Mohr-Coulomb
Unit Weight: Multiple Trial: 115 pcf
Constant Value: 115
Probabilistic: Normal(Mean=115,SD=10,Min=80,Max=145)
Cohesion: 0 psf
Phi: Multiple Trial: 32 °
Constant Value: 32
Probabilistic: Normal(Mean=32,SD=3,Min=22,Max=45)
Phi-B: 0 °

Bentonite Cutoff

Model: Mohr-Coulomb
Unit Weight: 95 pcf
Cohesion: 50 psf
Phi: 22 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (202.4033, 839.2801) ft
Left-Zone Right Coordinate: (228.97346, 842.633) ft
Left-Zone Increment: 8
Right Projection: Range
Right-Zone Left Coordinate: (249, 849) ft
Right-Zone Right Coordinate: (251, 849) ft
Right-Zone Increment: 8
Radius Increments: 8

Slip Surface Limits

Left Coordinate: (0, 838) ft
Right Coordinate: (500, 838) ft

Regions

	Material	Points	Area (ft^2)
Region 1	Bentonite Cutoff	5,21,19,9,3,4,10,20,22,6	42
Region 2	Blanket (CL)	28,27,23,7,19,21,30	433
Region 3	Levee Fill	7,1,2,8,20,10,4,3,9,19	351
Region 4	Blanket (CL)	22,20,8,24,25,26,14	593
Region 5	Blanket (CL)	13,11,27,28	150
Region 6	Sand Below Blanket (SP/SM)	15,13,28,29	800
Region 7	Sand Below Blanket (SP/SM)	29,28,30,21,5,6,22,14,26,12,16	7200
Region 8	Clay Below (SP/CL)	17,15,29,16,18	5000

Points

	X (ft)	Y (ft)
Point 1	241	849
Point 2	259	849
Point 3	249	847

Point 4	251	847
Point 5	249	826
Point 6	251	826
Point 7	224	840
Point 8	276	840
Point 9	249	840
Point 10	251	840
Point 11	0	838
Point 12	500	838
Point 13	0	836
Point 14	490	836
Point 15	0	820
Point 16	500	820
Point 17	0	810
Point 18	500	810
Point 19	249	838
Point 20	251	838
Point 21	249	836
Point 22	251	836
Point 23	164	838
Point 24	366	838
Point 25	409	838
Point 26	490	838
Point 27	100	838
Point 28	50	836
Point 29	50	820

Point 30	104.17741	836

Appendix C: Riverside Erosion Calculation Summaries

The software tool written by URS under contract to the Corps of Engineers was used to estimate erosion progression for each of the levee sections. Because of the armoring and very slow water velocities adjacent to the Ruby Wash Diversion Levee, no significant erosion is expected there. Below, the tables show the input parameters used for the analysis. Along with an estimate of the amount of erosion (feet) that would be anticipated along with the standard deviation of the anticipated erosion distribution. Winslow Station 29000 is also not anticipated to have significant erosion failure in the present channel configuration due to the armoring and because the main channel flow is away from the levee and the overall channel velocities are relatively small when compared to the main channel velocities for example at Winslow Station 37000.

As noted in the text of the report, a time integration is performed for the duration of the storm to determine the total distance the levee is eroded during an event. The average water elevation for the event was used in the analysis. Due to the very short peak, it was judged more appropriate to model the average storm conditions. Table C-1 Summarizes the peak water elevation, the average water elevation that was used in the analysis, and the calculated probability of unsatisfactory erosion performance for the Winslow Stations 51500 and 37000. Stream velocities at RWDL 495 and Winslow 29000 were determined to be too slow to cause catastrophic erosion and are not shown in the table below.

Table C-1 Peak Water Surface vs. Average Water Surface for Erosion Calculation

Location	Peak Water Elevation for Event (feet)	Approximate Average Water Elevation for Event (feet)	Approximate Probability of Unsatisfactory Performance
51500	4860.4	4855.0	0.01
	4862.3	4856.5	0.019
	4864.1	4857.0	0.02
37000	4849.7	4846.0	<0.01
	4850.9	4847.0	0.02
	4852.1	4847.5	0.06
	4853.5	4848.0	0.15

Figures C-1 and C-2 show the river stage vs. velocity information for Stations 51500 and 37000.

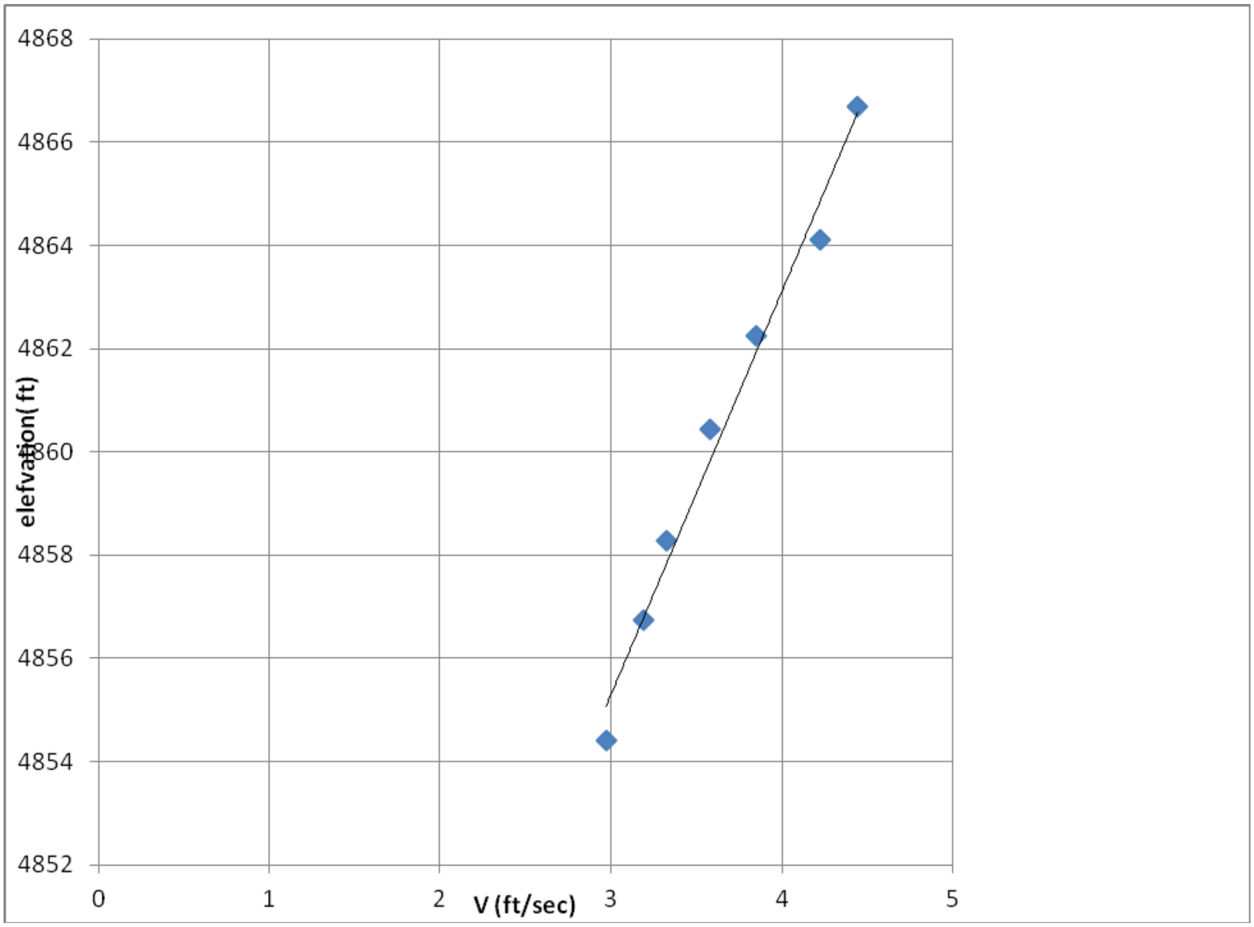


Figure C-1. Stream Velocity Rating Curve for Station 51500

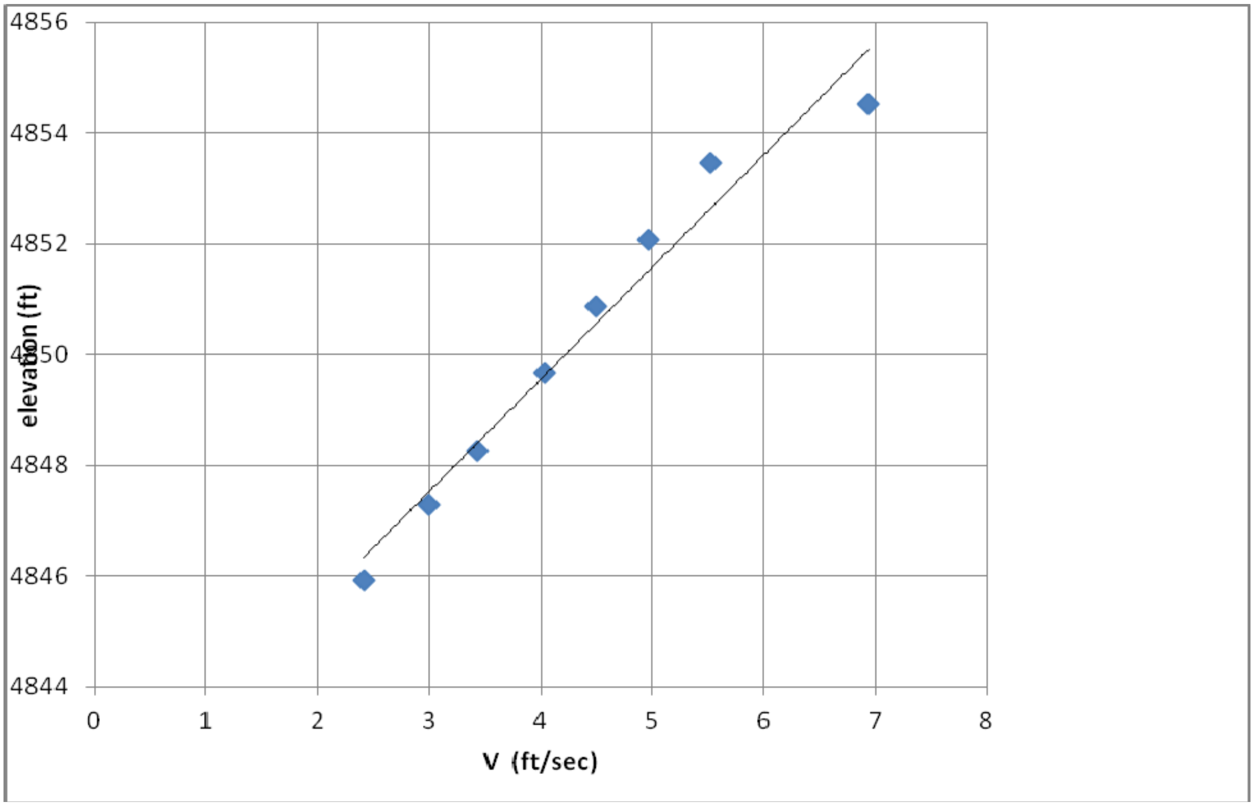
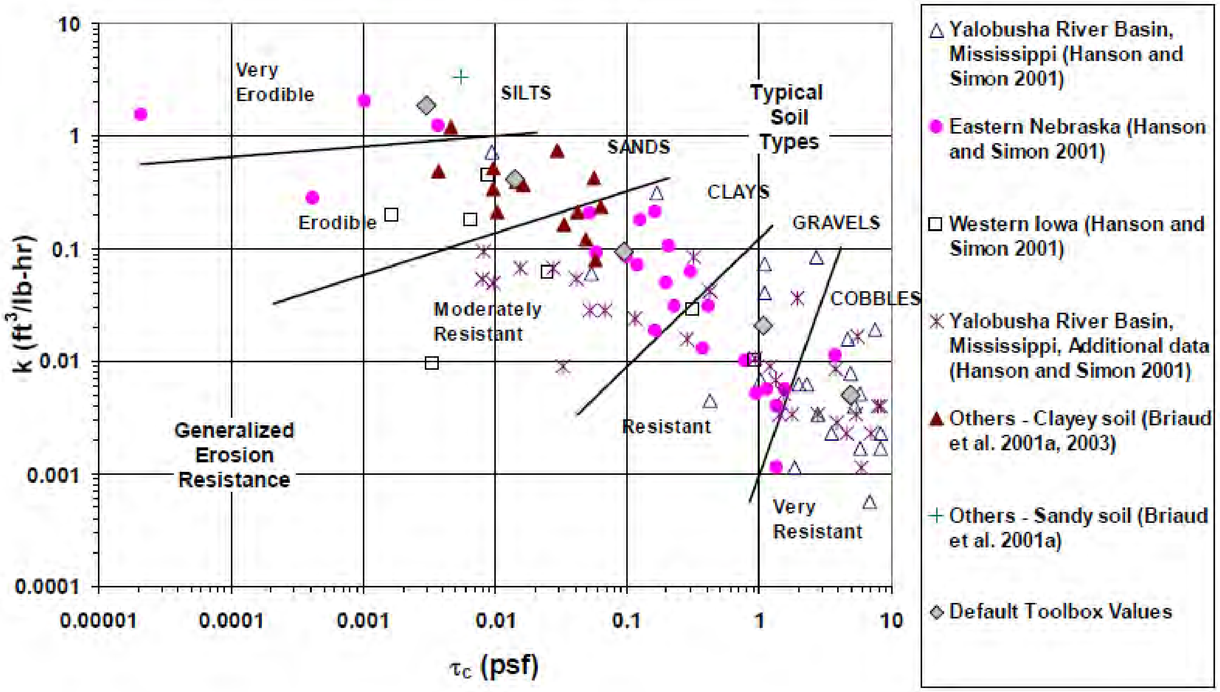


Figure C-2. Stream Velocity Rating Curve for Station 37000

Figure C-3 illustrates the basis for erodibility parameter selection.

Figure C-3 Erodibility Parameters



Levee/Foundation Material	ASTM Typical Soil Type	Critical Shear Stress, τ_c (psf)		Erodibility Coefficient, k (ft^3/lb-hr)	
		Mean	Coefficient of Variation	Mean	Coefficient of Variation
Very Resistant	Cobbles	4.869	0.446	0.005	0.952
Resistant	Gravel (GP-GW)	1.058	0.560	0.021	1.101
Moderately Resistant	CLAY (CL, CH, SC, GC)	0.094	0.917	0.094	0.800
Erodible	SAND (SP, SM and mixtures)	0.014	1.089	0.409	0.440
Very Erodible	SILT (ML)	0.003	0.785	1.867	0.473

Table C-2. Example Calculation Input for Winslow Station 37000 (25 yr event, average water level = 4846 feet)

1. Provide Project ID information

Levee ID	Winslow Levee		
Location	Station 37000		
Beginning Location (levee miles, river miles or stations)	Station 37000	Ending Location	Station 37000
Notes	Trial 1 -Partially Engineered/cutoffed Levee, No Armor, No Vegetation, Very Erodible Levee and Foundation, No Wind Waves, 10 to 100 yr floods		

2. Select Levee Type and Review/Edit Adjustment Factor for Levee Erosion Rate

Levee Type	Symbol	Default Adjustment Factor	User-Specified Adjustment Factor
L1 - Levees with Cutoffs	aL_1	1	

(Note: L1 levee type includes homogenous levees, levees with internal cutoff walls, zoned/partially engineered levees, and floodwalls type A)

3. Select Armor Category and Review/Edit Adjustment Factor for Levee and Foundation Erosion Rate

		Default Adjustment Factor		User-Specified Adjustment Factor	
Armor Category	Symbol	Critical armor velocity and wave height NOT exceeded	Critical armor velocity and/or wave height exceeded	Critical armor velocity and wave height NOT exceeded	Critical armor velocity and/or wave height exceeded
A1 - Armor present, Levee	aL_2_NE, aL_2_E	0	1		
A2 - Armor not present, Foundation	aF_2_NE, aF_2_E	1	1		

	Critical velocity at which armor fails (ft/s), CVL_a_L and CVL_a_F	Critical wave height at which armor fails (ft), Wcr_a_L and Wcr_a_F
... for levee	7	2
... for foundation	2	2

4. Select Vegetation Type and Review/Edit Adjustment Factor for Levee Erosion Rate

		Default Adjustment Factor		User-Specified Adjustment Factor	
Vegetation Type	Symbol	Critical vegetation velocity and wave height NOT exceeded	Critical vegetation velocity and/or wave height exceeded	Critical vegetation velocity and wave height NOT exceeded	Critical vegetation velocity and/or wave height exceeded
V3 - Neutral or None	aL_3_NE, aL_3_E	1	1		

Critical velocity at which veg. protection is lost (ft/s), CVL_v	Critical wave height at which veg. protection is lost (ft), Wcr_v
1	1

5. Select Levee Soil Type and Review/Edit Statistical Parameters of Erosion Soil Properties

		Default Values				User-Specified Values			
		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)	
Levee Soil Type	Symbol	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation
LSO3 - Moderately Resistant	τ_{cL}, k_L	0.094	0.917	0.094	0.800				

6. Select Foundation Soil Type and Review/Edit Statistical Parameters of Erosion Soil Properties

		Default Values				User-Specified Values			
		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)	
Foundation Soil Type	Symbol	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation
FSO5 - Very Erodible	τ_{cF}, k_F	0.003	0.785	1.867	0.473				

7. Provide Information on Relevant Channel, Levee, and Foundation Attributes

Effective levee width against erosion (ft)	W_E	12
Levee slope (X Horizontal to 1 Vertical; Specify X)	X_L	2
Foundation slope (X Horizontal to 1 Vertical; Specify X)	X_F	2
Landside toe elevation, NAVD 88 (ft)	LTE	4840
Maximum water surface elevation, NAVD 88 (ft)	LCE	4855
Channel bottom elevation, NAVD 88 (ft)	ZB	4838
Channel bottom width (ft)	B_W	1500
Bed roughness (ft)	k_b	0.0008
Levee is on a channel bend? (Yes or No), Bend If Yes, provide the following information.		Yes
- Radius of the bend (ft)	C_r	150
- Angle of the bend (degrees)	α	180

Levee Slope Category	Symbol	Default or User-Specified?	Default Adj. Factor	User-Specified Adj. Factor
LS2 - Steep (steeper than 2.5H to 1V)	aL_4	Default	1.2	
Foundation Slope Category	**Symbol**	**Default or User-Specified?**	**Default Adj. Factor**	**User-Specified Adj. Factor**
FS2 - Steep (steeper than 2.5H to 1V)	aF_1	Default	1.2	

8. Characterize Vulnerability to Wind/Wave Erosion

Is the levee location vulnerable to wind/wave erosion impact?	Wind	No
If the answer is No, skip the rest of this step. If the answer is Yes, provide the following information:		
Levee slope roughness (ft)	k_l	0.0008

Maximum wind speed against the levee face (miles/hour)	Mean	Coefficient of Variation	Upperbound, Upper_U_pi
	50	0.2	30
Maximum fetch length (ft)		F	600
Duration of Wind (hrs)		d_w	2
Efficiency of wave breaking to erode sediment		Eff	7.50%

9. Current Speed

	Water Surface Elevation, NAVD 88 (ft)	Velocity (ft/sec)	Duration, d (hours)	Velocity based on Mannings coeff. (ft/sec)
	4846	2.4	84	6.0
	4859	3.1	0	11.3
	4859	3	0	11.3
TOL	4855	2.52	0	9.9

Note - To avoid divide by zero errors when using this tool for screening (i.e., WSE = Top of Levee), input ascending water surface elevations, top of levee velocities and top of levee durations in the above table, even if data is unavailable for the lower water surfaces. For a full Probability of Failure versus WSE table and graph, complete velocity and duration versus WSE values must be input.

Optional Input
Mannings coefficient equivalent velocity calculator
(Note: For information only, not needed for computations)

Channel bottom slope (X Hori. to 1 Vert.; Specify X), X_CB	600
Mannings coefficient for channel bottom, n	0.04

Table C-3 Example Intermediate Calculation for 1 trial (of 1000 trials per water level evaluated) Station 37000, 25-year event

Simulation Trial #	Simulated Value of Wind Speed, U' (miles/hr)	Wind-stress Factor, U_a (ft/sec)	Simulated Value of Levee Erodibility Coefficient, k_L (ft^3/lb-hr)	Simulated Value of Foundation Erodibility Coefficient, k_F (ft^3/lb-hr)	Simulated Value of Levee Critical Sheer Stress, τ_{CL} (lb/ft^2)	Simulated Value of Foundation Critical Sheer Stress, τ_{CF} (lb/ft^2)	Average Period of Wave, T (sec)	Water Surface Elevation, NAVD 88 (ft)	Water Surafce Elevation above Channel Bottom, h (ft)	Wave Height, H (ft)	Wave Length, L (ft)	Horizontal Mean Wave Orbital Motion, a (ft)	Wave Friction Factor, f_w (unitless)	Horizontal Mean Orbital Wave Velocity, U_b (ft/sec)	Bottom Current Sheer Stress on Levee due to Wind Waves, τ_{LW} (lb/ft^2)	Bottom Current Sheer Stress on Foundation due to Wind Waves, τ_{FW} (lb/ft^2)
1	80	189.6687	0.0730812	1.3624104	0.044048	0.0019828	1.36864	4846	8	0	0	0	0	0	0	0
1	80	189.6687	0.0730812	1.3624104	0.044048	0.0019828	1.36864	4850.333	12.33333	0	0	0	0	0	0	0
1	80	189.6687	0.0730812	1.3624104	0.044048	0.0019828	1.36864	4854.667	16.66667	0	0	0	0	0	0	0
1	80	189.6687	0.0730812	1.3624104	0.044048	0.0019828	1.36864	4859	21	0	0	0	0	0	0	0
1	80	189.6687	0.0730812	1.3624104	0.044048	0.0019828	1.36864	4859	21	0	0	0	0	0	0	0
1	80	189.6687	0.0730812	1.3624104	0.044048	0.0019828	1.36864	4859	21	0	0	0	0	0	0	0
1	80	189.6687	0.0730812	1.3624104	0.044048	0.0019828	1.36864	4859	21	0	0	0	0	0	0	0
1	80	189.6687	0.0730812	1.3624104	0.044048	0.0019828	1.36864	4857.667	19.66667	0	0	0	0	0	0	0
1	80	189.6687	0.0730812	1.3624104	0.044048	0.0019828	1.36864	4856.333	18.33333	0	0	0	0	0	0	0
1	80	189.6687	0.0730812	1.3624104	0.044048	0.0019828	1.36864	4855	17	0	0	0	0	0	0	0

Current Speed, V (ft/sec)	Bend Factor, x (unitless)	Adjusted Current Speed, V' (ft/sec)	Current Friction Factor - Levee, f_{CL} (unitless)	Current Friction Factor - Foundation, f_{CF} (unitless)	Sheer Stress on Levee due to Velcotiy, τ_{LV} (lb/ft^2)	Sheer Stress on Foundation due to Velcotiy, τ_{FV} (lb/ft^2)	Wave Height at Breaking, h_b (ft)	Adjusted Water Depth for Wave Model, h' (ft)	Wave Breaking - Levee? (Y/N)	Sheer Stress on Levee due to Wave Breaking, τ_{LB} (lb/ft^2)	Wave Breaking - Foundation ? (Y/N)	Sheer Stress on Foundation due to Wave Breaking, τ_{FB} (lb/ft^2)
2.4	0.101991	3.234854	0.002373	0.002373	0.024084	0.024084	0	0	0	0	0	0
2.633333	0.065416	3.742671	0.002206	0.002206	0.029963	0.029963	0	0	0	0	0	0
2.866667	0.047872	4.208517	0.0021	0.0021	0.036061	0.036061	0	0	0	0	0	0
3	0.037578	4.501584	0.002023	0.002023	0.039756	0.039756	0	0	0	0	0	0
3	0.037578	4.501584	0.002023	0.002023	0.039756	0.039756	0	0	0	0	0	0
3	0.037578	4.501584	0.002023	0.002023	0.039756	0.039756	0	0	0	0	0	0
3	0.037578	4.501584	0.002023	0.002023	0.039756	0.039756	0	0	0	0	0	0
3.028205	0.040261	4.517056	0.002044	0.002044	0.040451	0.040451	0	0	0	0	0	0
2.95641	0.043336	4.381009	0.002068	0.002068	0.038481	0.038481	0	0	0	0	0	0
2.884615	0.046894	4.243255	0.002093	0.002093	0.036542	0.036542	0	0	0	0	0	0

aL_1	aL_2	aL_3	aL_4	aF_1	aF_2	TAF_levee	TAF_foundation	Duration for Velocity, d (hours)	Levee Erosion due to Wave Bottom Current & Wave Breaking, ε_{LWB} (ft)	Foundation Erosion due to Wave Bottom Current & Wave Breaking, ε_{FWB} (ft)	Levee Erosion due to Velocity, ε_{LV} (ft)	Foundation Erosion due to Velocity, ε_{FV} (ft)	Total Levee Erosion, ε_{LT} (ft)	Total Foundation Erosion, ε_{FT} (ft)	$[\varepsilon_{LT} - mean(\varepsilon_{LT})]^2$	$[\varepsilon_{FT} - mean(\varepsilon_{FT})]^2$
1	0	1	1.2	1.2	1	0	1.2	84	0	0	0	3.0351873	0	3.0351873	0	0.743783
1	0	1	1.2	1.2	1	0	1.2	56	0	0	0	2.5617455	0	2.5617455	0	0.571119
1	0	1	1.2	1.2	1	0	1.2	28	0	0	0	1.5600059	0	1.5600059	0	0.222409
1	0	1	1.2	1.2	1	0	1.2	0	0	0	0	0	0	0	0	0
1	0	1	1.2	1.2	1	0	1.2	0	0	0	0	0	0	0	0	0
1	0	1	1.2	1.2	1	0	1.2	0	0	0	0	0	0	0	0	0
1	0	1	1.2	1.2	1	0	1.2	0	0	0	0	0	0	0	0	0
1	0	1	1.2	1.2	1	0	1.2	8.615385	0	0	0	0.5418279	0	0.5418279	0	0.027513
1	0	1	1.2	1.2	1	0	1.2	17.23077	0	0	0	1.0281805	0	1.0281805	0	0.09804
1	0	1	1.2	1.2	1	0	1.2	25.84615	0	0	0	1.4603074	0	1.4603074	0	0.195491

Table C-4 Winslow Station 37000 Estimated Erosion for a Storm with a Peak Water Elevation of 4849.7 feet (average elevation of 4846)

Levee ID	Winslow Levee		
Location	Station 37000		
Beginning	Station 37000	**Ending**	Station 37000
Notes	Trial 1 -Partially Engineered/cutoffed Levee, Armor, No Vegetation, Very Erodible Levee and Foundation, No Wind Waves, 10 to 100 yr floods		

Run Time: *3/23/2012 12:34*

				Levee Erosion (ft)		Foundation Erosion (ft)	
Water Surface Elevation, NAVD 88 (ft)	**Probability of Levee Erosion Failure**	**Probability of Foundation Erosion Failure**	**Total Probability of Erosion Failure**	**Mean**	**Standard Deviation**	**Mean**	**Standard Deviation**
4846	0	0.005	0.005	0.000	0.000	4.014	2.020

Table C-5 Station 37000 Erosion inputs 50 year event (average water elevation = 4847 feet)

1. Provide Project ID Information

Levee ID	Winslow Levee		
Location	Station 37000		
Beginning Location (levee miles, river miles or stations)	Station 37000	Ending Location	Station 37000
Notes	Trial 1 -Partially Engineered/cutoffed Levee, Armor, No Vegetation, Very Erodible Levee and Foundation, No Wind Waves; 10 to 100 yr floods		

2. Select Levee Type and Review/Edit Adjustment Factor for Levee Erosion Rate

Levee Type	Symbol	Default Adjustment Factor	User-Specified Adjustment Factor
L1 - Levees with Cutoffs	aL_1	1	

(Note: L1 levee type includes homogenous levees, levees with internal cutoff walls, zoned/partially engineered levees, and floodwalls type A)

3. Select Armor Category and Review/Edit Adjustment Factor for Levee and Foundation Erosion Rate

		Default Adjustment Factor		User-Specified Adjustment Factor	
Armor Category	Symbol	Critical armor velocity and wave height NOT exceeded	Critical armor velocity and/or wave height exceeded	Critical armor velocity and wave height NOT exceeded	Critical armor velocity and/or wave height exceeded
A1 - Armor present, Levee	aL_2_NE, aL_2_E	0	1		
A2 - Armor not present, Foundation	aF_2_NE, aF_2_E	1	1		

	Critical velocity at which armor fails (ft/s), CVL_a_L and CVL_a_F	Critical wave height at which armor fails (ft), Wcr_a_L and Wcr_a_F
... for levee	7	2
... for foundation	2	2

4. Select Vegetation Type and Review/Edit Adjustment Factor for Levee Erosion Rate

		Default Adjustment Factor		User-Specified Adjustment Factor	
Vegetation Type	Symbol	Critical vegetation velocity and wave height NOT exceeded	Critical vegetation velocity and/or wave height exceeded	Critical vegetation velocity and wave height NOT exceeded	Critical vegetation velocity and/or wave height exceeded
V3 - Neutral or None	aL_3_NE, aL_3_E	1	1		

Critical velocity at which veg. protection is lost (ft/s), CVL_v	Critical wave height at which veg. protection is lost (ft), Wcr_v
1	1

5. Select Levee Soil Type and Review/Edit Statistical Parameters of Erosion Soil Properties

		Default Values				User-Specified Values			
		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)	
Levee Soil Type	Symbol	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation
LSO3 - Moderately Resistant	τ_{cL}, k_L	0.094	0.917	0.094	0.800				

6. Select Foundation Soil Type and Review/Edit Statistical Parameters of Erosion Soil Properties

		Default Values				User-Specified Values			
		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)	
Foundation Soil Type	Symbol	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation
FSO5 - Very Erodible	τ_{cF}, k_F	0.003	0.785	1.867	0.473				

7. Provide Information on Relevant Channel, Levee, and Foundation Attributes

Effective levee width against erosion (ft)	W_E	12
Levee slope (X Horizontal to 1 Vertical; Specify X)	X_L	2
Foundation slope (X Horizontal to 1 Vertical; Specify X)	X_F	2
Landside toe elevation, NAVD 88 (ft)	LTE	4840
Maximum water surface elevation, NAVD 88 (ft)	LCE	4855
Channel bottom elevation, NAVD 88 (ft)	ZB	4838
Channel bottom width (ft)	B_w	1500
Bed roughness (ft)	k_s	0.0008
Levee is on a channel bend? (Yes or No), Bend If Yes, provide the following information.		Yes
- Radius of the bend (ft)	C_r	150
- Angle of the bend (degrees)	α	180

Levee Slope Category	Symbol	Default or User-Specified?	Default Adj. Factor	User-Specified Adj. Factor
LS2 - Steep (steeper than 2.5H to 1V)	aL_4	Default	1.2	

Foundation Slope Category	Symbol	Default or User-Specified?	Default Adj. Factor	User-Specified Adj. Factor
FS2 - Steep (steeper than 2.5H to 1V)	aF_1	Default	1.2	

8. Characterize Vulnerability to Wind/Wave Erosion

Is the levee location vulnerable to wind/wave erosion impact?	Wind	No
If the answer is No, skip the rest of this step. If the answer is Yes, provide the following information:		
Levee slope roughness (ft)	k_l	[illegible]

	Mean	Coefficient of Variation	Upperbound, Upper_U_pi
Maximum wind speed against the levee face (miles/hour)	56	0.5	30
Maximum fetch length (ft)		F	500
Duration of Wind (hrs)		d_w	2
Efficiency of wave breaking to erode sediment		Eff	[illegible]

9. Current Speed

	Water Surface Elevation, NAVD 88 (ft)	Velocity (ft/sec)	Duration, d (hours)	Velocity based on Mannings coeff. (ft/sec)
	4847	2.7	84	6.5
	4859	3.1	0	11.3
	4859	3	0	11.1
TOL	4855	2.52	0	9.9

Note - To avoid divide by zero errors when using this tool for screening (i.e., WSE = Top of Levee), input ascending water surface elevations, top of levee velocities and top of levee durations in the above table, even if data is unavailable for the lower water surfaces. For a full Probability of Failure versus WSE table and graph, complete velocity and duration versus WSE values must be input.

Optional Input
Mannings coefficient equivalent velocity calculator
(Note: For information only, not needed for computations)

Channel bottom slope (X Hori. to 1 Vert.; Specify X), X_CB	600
Mannings coefficient for channel bottom, n	0.04

Table C-6 Station 37000 Erosion Output 50 year event (average water elevation = 4847 feet)

Levee ID	Winslow Levee		
Location	Station 37000		
Beginning	Station 37000	**Ending**	Station 37000
Notes	Trial 1 -Partially Engineered/cutoffed Levee, Armor, No Vegetation, Very Erodible Levee and Foundation, No Wind Waves, 10 to 100 yr floods		

Run Time: *3/23/2012 12:44*

				Levee Erosion (ft)		Foundation Erosion (ft)	
Water Surface Elevation, NAVD 88 (ft)	**Probability of Levee Erosion Failure**	**Probability of Foundation Erosion Failure**	**Total Probability of Erosion Failure**	**Mean**	**Standard Deviation**	**Mean**	**Standard Deviation**
4847	0	0.02	0.020	0.000	0.000	5.225	2.403

Table C-7 Station 37000 Erosion inputs 100-year event (average water elevation = 4847.5 feet)

1. Provide Project ID information

Levee ID	Winslow Levee		
Location	Station 37000		
Beginning Location (levee miles, river miles or stations)	Station 37000	Ending Location	Station 37000
Notes	Trial 1 -Partially Engineered/cutoffed Levee, Armor, No Vegetation, Very Erodible Levee and Foundation, No Wind Waves, 10 to 100 yr floods		

2. Select Levee Type and Review/Edit Adjustment Factor for Levee Erosion Rate

Levee Type	Symbol	Default Adjustment Factor	User-Specified Adjustment Factor
L1 - Levees with Cutoffs	aL_1	1	

(Note: L1 levee type includes homogenous levees, levees with internal cutoff walls, zoned/partially engineered levees, and floodwalls type A)

3. Select Armor Category and Review/Edit Adjustment Factor for Levee and Foundation Erosion Rate

		Default Adjustment Factor		User-Specified Adjustment Factor	
Armor Category	Symbol	Critical armor velocity and wave height NOT exceeded	Critical armor velocity and/or wave height exceeded	Critical armor velocity and wave height NOT exceeded	Critical armor velocity and/or wave height exceeded
A1 - Armor present, Levee	aL_2_NE, aL_2_E	0	1		
A2 - Armor not present, Foundation	aF_2_NE, aF_2_E	1	1		

	Critical velocity at which armor fails (ft/s), CVL_a_L and CVL_a_F	Critical wave height at which armor fails (ft), Wcr_a_L and Wcr_a_F
... for levee	7	2
... for foundation	2	2

4. Select Vegetation Type and Review/Edit Adjustment Factor for Levee Erosion Rate

		Default Adjustment Factor		User-Specified Adjustment Factor	
Vegetation Type	Symbol	Critical vegetation velocity and wave height NOT exceeded	Critical vegetation velocity and/or wave height exceeded	Critical vegetation velocity and wave height NOT exceeded	Critical vegetation velocity and/or wave height exceeded
V3 - Neutral or None	aL_3_NE, aL_3_E	1	1		

Critical velocity at which veg. protection is lost (ft/s), CVL_v	Critical wave height at which veg. protection is lost (ft), Wcr_v
1	1

5. Select Levee Soil Type and Review/Edit Statistical Parameters of Erosion Soil Properties

		Default Values				User-Specified Values			
		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)	
Levee Soil Type	Symbol	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation
LSO3 - Moderately Resistant	τ_{CL}, k_L	0.094	0.917	0.094	0.800				

6. Select Foundation Soil Type and Review/Edit Statistical Parameters of Erosion Soil Properties

		Default Values				User-Specified Values			
		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)	
Foundation Soil Type	Symbol	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation
FSO5 - Very Erodible	τ_{CF}, k_F	0.003	0.785	1.867	0.473				

7. Provide Information on Relevant Channel, Levee, and Foundation Attributes

Effective levee width against erosion (ft)	W_E	12
Levee slope (X Horizontal to 1 Vertical; Specify X)	X_L	2
Foundation slope (X Horizontal to 1 Vertical; Specify X)	X_F	2
Landside toe elevation, NAVD 88 (ft)	LTE	4840
Maximum water surface elevation, NAVD 88 (ft)	LCE	4855
Channel bottom elevation, NAVD 88 (ft)	ZB	4838
Channel bottom width (ft)	B_W	1500
Bed roughness (ft)	k_b	0.0008
Levee is on a channel bend? (Yes or No), Bend If Yes, provide the following information.		Yes
- Radius of the bend (ft)	C_r	150
- Angle of the bend (degrees)	α	180

Levee Slope Category	Symbol	Default or User-Specified?	Default Adj. Factor	User-Specified Adj. Factor
LS2 - Steep (steeper than 2.5H to 1V)	aL_4	Default	1.2	

Foundation Slope Category	Symbol	Default or User-Specified?	Default Adj. Factor	User-Specified Adj. Factor
FS2 - Steep (steeper than 2.5H to 1V)	aF_1	Default	1.2	

8. Characterize Vulnerability to Wind/Wave Erosion

Is the levee location vulnerable to wind/wave erosion impact?	Wind	No
If the answer is No, skip the rest of this step. If the answer is Yes, provide the following information:		
Levee slope roughness (ft)	k_l	0.0003

	Mean	Coefficient of Variation	Upperbound, Upper_U_pi
Maximum wind speed against the levee face (miles/hour)	50	0.5	80

Maximum fetch length (ft)	F	500
Duration of Wind (hrs)	d_w	2
Efficiency of wave breaking to erode sediment	Eff	7.50%

9. Current Speed

	Water Surface Elevation, NAVD 88 (ft)	Velocity (ft/sec)	Duration, d (hours)	Velocity based on Mannings coeff. (ft/sec)
	4847.5	3	84	6.8
	4859	3.1	0	11.3
	4859	3	0	11.3
TOL	4855	2.52	0	9.9

Optional Input
Mannings coefficient equivalent velocity calculator
(Note: For information only, not needed for computations)

Channel bottom slope (X Hori. to 1 Vert.; Specify X), X_CB	600
Mannings coefficient for channel bottom, n	0.04

Table C-8 Station 37000 Erosion Output 100-year event (average water elevation = 4847.5 feet)

Levee ID	Winslow Levee		
Location	Station 37000		
Beginning	Station 37000	**Ending**	Station 37000
Notes	Trial 1 -Partially Engineered/cutoffed Levee, Armor, No Vegetation, Very Erodible Levee and Foundation, No Wind Waves, 10 to 100 yr floods		

Run Time: *3/23/2012 12:52*

				Levee Erosion (ft)		Foundation Erosion (ft)	
Water Surface Elevation, NAVD 88 (ft)	**Probability of Levee Erosion Failure**	**Probability of Foundation Erosion Failure**	**Total Probability of Erosion Failure**	**Mean**	**Standard Deviation**	**Mean**	**Standard Deviation**
4847.5	0	0.056	0.056	0.000	0.000	6.542	3.110

Table C-9 Station 37000 Erosion inputs 200-year event (average water elevation = 4848.0 feet)

1. Provide Project ID information

Levee ID	Winslow Levee		
Location	Station 37000		
Beginning Location (levee miles, river miles or stations)	Station 37000	Ending Location	Station 37000
Notes	Trial 1 -Partially Engineered/cutoffed Levee, Armor, No Vegetation, Very Erodible Levee and Foundation, No Wind Waves, 10 to 100 yr floods		

2. Select Levee Type and Review/Edit Adjustment Factor for Levee Erosion Rate

Levee Type	Symbol	Default Adjustment Factor	User-Specified Adjustment Factor
L1 - Levees with Cutoffs	aL_1	1	

(Note: L1 levee type includes homogenous levees, levees with internal cutoff walls, zoned/partially engineered levees, and floodwalls type A)

3. Select Armor Category and Review/Edit Adjustment Factor for Levee and Foundation Erosion Rate

Armor Category	Symbol	Default Adjustment Factor		User-Specified Adjustment Factor	
		Critical armor velocity and wave height NOT exceeded	Critical armor velocity and/or wave height exceeded	Critical armor velocity and wave height NOT exceeded	Critical armor velocity and/or wave height exceeded
A1 - Armor present, Levee	aL_2_NE, aL_2_E	0	1		
A2 - Armor not present, Foundation	aF_2_NE, aF_2_E	1	1		

	Critical velocity at which armor fails (ft/s), CVL_a_L and CVL_a_F	Critical wave height at which armor fails (ft), Wcr_a_L and Wcr_a_F
... for levee	7	2
... for foundation	2	2

4. Select Vegetation Type and Review/Edit Adjustment Factor for Levee Erosion Rate

Vegetation Type	Symbol	Default Adjustment Factor		User-Specified Adjustment Factor	
		Critical vegetation velocity and wave height NOT exceeded	Critical vegetation velocity and/or wave height exceeded	Critical vegetation velocity and wave height NOT exceeded	Critical vegetation velocity and/or wave height exceeded
V3 - Neutral or None	aL_3_NE, aL_3_E	1	1		

Critical velocity at which veg. protection is lost (ft/s), CVL_v	Critical wave height at which veg. protection is lost (ft), Wcr_v
1	1

5. Select Levee Soil Type and Review/Edit Statistical Parameters of Erosion Soil Properties

Levee Soil Type	Symbol	Default Values: Critical Shear Stress (psf) Mean	Default Values: Critical Shear Stress (psf) Coefficient of Variation	Default Values: Erodibility Coefficient (ft^3/lb-hr) Mean	Default Values: Erodibility Coefficient (ft^3/lb-hr) Coefficient of Variation	User-Specified Values: Critical Shear Stress (psf) Mean	User-Specified Values: Critical Shear Stress (psf) Coefficient of Variation	User-Specified Values: Erodibility Coefficient (ft^3/lb-hr) Mean	User-Specified Values: Erodibility Coefficient (ft^3/lb-hr) Coefficient of Variation
LSO3 - Moderately Resistant	τ_{CL}, k_L	0.094	0.917	0.094	0.800				

6. Select Foundation Soil Type and Review/Edit Statistical Parameters of Erosion Soil Properties

Foundation Soil Type	Symbol	Default Values: Critical Shear Stress (psf) Mean	Default Values: Critical Shear Stress (psf) Coefficient of Variation	Default Values: Erodibility Coefficient (ft^3/lb-hr) Mean	Default Values: Erodibility Coefficient (ft^3/lb-hr) Coefficient of Variation	User-Specified Values: Critical Shear Stress (psf) Mean	User-Specified Values: Critical Shear Stress (psf) Coefficient of Variation	User-Specified Values: Erodibility Coefficient (ft^3/lb-hr) Mean	User-Specified Values: Erodibility Coefficient (ft^3/lb-hr) Coefficient of Variation
FSO5 - Very Erodible	τ_{CF}, k_F	0.003	0.785	1.867	0.473				

7. Provide Information on Relevant Channel, Levee, and Foundation Attributes

Effective levee width against erosion (ft)	W_E	12
Levee slope (X Horizontal to 1 Vertical; Specify X)	X_L	2
Foundation slope (X Horizontal to 1 Vertical; Specify X)	X_F	2
Landside toe elevation, NAVD 88 (ft)	LTE	4840
Maximum water surface elevation, NAVD 88 (ft)	LCE	4855
Channel bottom elevation, NAVD 88 (ft)	ZB	4838
Channel bottom width (ft)	B_W	1500
Bed roughness (ft)	k_s	0.0008
Levee is on a channel bend? (Yes or No), Bend If Yes, provide the following information.		Yes
- Radius of the bend (ft)	C_r	150
- Angle of the bend (degrees)	α	180

Levee Slope Category	Symbol	Default or User-Specified?	Default Adj. Factor	User-Specified Adj. Factor
LS2 - Steep (steeper than 2.5H to 1V)	aL_4	Default	1.2	

Foundation Slope Category	Symbol	Default or User-Specified?	Default Adj. Factor	User-Specified Adj. Factor
FS2 - Steep (steeper than 2.5H to 1V)	aF_1	Default	1.2	

8. Characterize Vulnerability to Wind/Wave Erosion

Is the levee location vulnerable to wind/wave erosion impact?	Wind	No
If the answer is No, skip the rest of this step. If the answer is Yes, provide the following information:		
Levee slope roughness (ft)	k_l	0.0008

	Mean	Coefficient of Variation	Upperbound, Upper_U_pi
Maximum wind speed against the levee face (miles/hour)	50	0.6	60
Maximum fetch length (ft)		F	600
Duration of Wind (hrs)		d_w	2
Efficiency of wave breaking to erode sediment		Eff	7.50%

9. Current Speed

	Water Surface Elevation, NAVD 88 (ft)	Velocity (ft/sec)	Duration, d (hours)	Velocity based on Mannings coeff. (ft/sec)
	4848	3.3	84	7.0
	4859	3.1	0	11.3
	4859	3	0	11.3
TOL	4855	2.52	0	9.9

Optional Input
Mannings coefficient equivalent velocity calculator
(Note: For information only, not needed for computations)

Channel bottom slope (X Hori. to 1 Vert.; Specify X), X_CB	600
Mannings coefficient for channel bottom, n	0.04

Table C-10 Station 37000 Erosion Output 200-year event (average water elevation = 4848.0 feet)

Levee ID	Winslow Levee		
Location	Station 37000		
Beginning	Station 37000	**Ending**	Station 37000
Notes	Trial 1 -Partially Engineered/cutoffed Levee, Armor, No Vegetation, Very Erodible Levee and Foundation, No Wind Waves, 10 to 100 yr floods		

Run Time: *3/23/2012 12:57*

				Levee Erosion (ft)		Foundation Erosion (ft)	
Water Surface Elevation, NAVD 88 (ft)	**Probability of Levee Erosion Failure**	**Probability of Foundation Erosion Failure**	**Total Probability of Erosion Failure**	**Mean**	**Standard Deviation**	**Mean**	**Standard Deviation**
4848	0	0.156	0.156	0.000	0.000	8.237	3.860

Table C-11 Station 515000 Erosion inputs 25-year event (average water elevation = 4855 feet)

1. Provide Project ID information

Levee ID	Winslow Levee		
Location	Station 51500		
Beginning Location (levee miles, river miles or stations)	Station 51500	Ending Location	Station 51500
Notes	Trial 1 -Partially Engineered/cutoffed Levee, No Armor, No Vegetation, Very Erodible Levee and Foundation, No Wind Waves, 10 to 100 yr floods		

2. Select Levee Type and Review/Edit Adjustment Factor for Levee Erosion Rate

Levee Type	Symbol	Default Adjustment Factor	User-Specified Adjustment Factor
L1 - Levees with Cutoffs	aL_1	1	

(Note: L1 levee type includes homogenous levees, levees with internal cutoff walls, zoned/partially engineered levees, and floodwalls type A)

3. Select Armor Category and Review/Edit Adjustment Factor for Levee and Foundation Erosion Rate

Armor Category	Symbol	Default Adjustment Factor: Critical armor velocity and wave height NOT exceeded	Default Adjustment Factor: Critical armor velocity and/or wave height exceeded	User-Specified Adjustment Factor: Critical armor velocity and wave height NOT exceeded	User-Specified Adjustment Factor: Critical armor velocity and/or wave height exceeded
A2 - Armor not present, Levee	aL_2_NE, aL_2_E	1	1		
A2 - Armor not present, Foundation	aF_2_NE, aF_2_E	1	1		

	Critical velocity at which armor fails (ft/s), CVL_a_L and CVL_a_F	Critical wave height at which armor fails (ft), Wcr_a_L and Wcr_a_F
... for levee	2	2
... for foundation	2	2

4. Select Vegetation Type and Review/Edit Adjustment Factor for Levee Erosion Rate

Vegetation Type	Symbol	Default Adjustment Factor: Critical vegetation velocity and wave height NOT exceeded	Default Adjustment Factor: Critical vegetation velocity and/or wave height exceeded	User-Specified Adjustment Factor: Critical vegetation velocity and wave height NOT exceeded	User-Specified Adjustment Factor: Critical vegetation velocity and/or wave height exceeded
V3 - Neutral or None	aL_3_NE, aL_3_E	1	1		

Critical velocity at which veg. protection is lost (ft/s), CVL_v	Critical wave height at which veg. protection is lost (ft), Wcr_v
1	1

5. Select Levee Soil Type and Review/Edit Statistical Parameters of Erosion Soil Properties

		Default Values				User-Specified Values			
		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)	
Levee Soil Type	Symbol	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation
LSO5 - Very Erodible	τ_{cL}, k_L	0.003	0.785	1.867	0.473				

6. Select Foundation Soil Type and Review/Edit Statistical Parameters of Erosion Soil Properties

		Default Values				User-Specified Values			
		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)	
Foundation Soil Type	Symbol	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation
FSO5 - Very Erodible	τ_{cF}, k_F	0.003	0.785	1.867	0.473				

7. Provide Information on Relevant Channel, Levee, and Foundation Attributes

Effective levee width against erosion (ft)	W_E	10
Levee slope (X Horizontal to 1 Vertical; Specify X)	X_L	2
Foundation slope (X Horizontal to 1 Vertical; Specify X)	X_F	2
Landside toe elevation, NAVD 88 (ft)	LTE	4854
Maximum water surface elevation, NAVD 88 (ft)	LCE	4860
Channel bottom elevation, NAVD 88 (ft)	ZB	4848
Channel bottom width (ft)	B_W	450
Bed roughness (ft)	k_b	0.0008
Levee is on a channel bend? (Yes or No), Bend If Yes, provide the following information.		No
- Radius of the bend (ft)	C_r	10000
- Angle of the bend (degrees)	α	10

Levee Slope Category	Symbol	Default or User-Specified?	Default Adj. Factor	User-Specified Adj. Factor
LS2 - Steep (steeper than 2.5H to 1V)	aL_4	Default	1.2	

Foundation Slope Category	Symbol	Default or User-Specified?	Default Adj. Factor	User-Specified Adj. Factor
FS2 - Steep (steeper than 2.5H to 1V)	aF_1	Default	1.2	

8. Characterize Vulnerability to Wind/Wave Erosion

Is the levee location vulnerable to wind/wave erosion impact?	Wind	No
If the answer is No, skip the rest of this step. If the answer is Yes, provide the following information:		
Levee slope roughness (ft)	k_l	0.0003

Maximum wind speed against the levee face (miles/hour)	Mean	Coefficient of Variation	Upperbound, Upper_U_pi
	10	0.5	30

Maximum fetch length (ft)	F	600
Duration of Wind (hrs)	d_w	2
Efficiency of wave breaking to erode sediment	Eff	7.50%

9. Current Speed

	Water Surface Elevation, NAVD 88 (ft)	Velocity (ft/sec)	Duration, d (hours)	Velocity based on Mannings coeff. (ft/sec)
	4855	2.9	84	5.4
	4857	3.5	0	6.4
	4859	3.5	0	7.2
TOL	4860	3.5	0	7.7

Optional Input
Mannings coefficient equivalent velocity calculator
(Note: For information only, not needed for computations)

Channel bottom slope (X Hori. to 1 Vert.; Specify X), X_CB	600
Mannings coefficient for channel bottom, n	0.04

Table C-12 Station 515000 Erosion Output 25-year event (average water elevation = 4855 feet)

Levee ID	Winslow Levee		
Location	Station 51500		
Beginning	Station 51500	**Ending**	Station 51500
Notes	Trial 1 -Partially Engineered/cutoffed Levee, No Armor, No Vegetation, Very Erodible Levee and Foundation, No Wind Waves, 10 to 100 yr floods		

Run Time: *3/23/2012 13:11*

				Levee Erosion (ft)		Foundation Erosion (ft)	
Water Surface Elevation, NAVD 88 (ft)	**Probability of Levee Erosion Failure**	**Probability of Foundation Erosion Failure**	**Total Probability of Erosion Failure**	**Mean**	**Standard Deviation**	**Mean**	**Standard Deviation**
4855	0.003	0.005	0.008	3.150	1.590	3.209	1.622

Table C-13 Station 515000 Erosion inputs 50-year event (average water elevation = 4856.5 feet)

1. Provide Project ID information

Levee ID	Winslow Levee		
Location	Station 51500		
Beginning Location (levee miles, river miles or stations)	Station 51500	Ending Location	Station 51500
Notes	Trial 1 -Partially Engineered/cutoffed Levee, No Armor, No Vegetation, Very Erodible Levee and Foundation, No Wind Waves, 10 to 100 yr floods		

2. Select Levee Type and Review/Edit Adjustment Factor for Levee Erosion Rate

Levee Type	Symbol	Default Adjustment Factor	User-Specified Adjustment Factor
L1 - Levees with Cutoffs	aL_1	1	

(Note: L1 levee type includes homogenous levees, levees with internal cutoff walls, zoned/partially engineered levees, and floodwalls type A)

3. Select Armor Category and Review/Edit Adjustment Factor for Levee and Foundation Erosion Rate

Armor Category	Symbol	Default Adjustment Factor: Critical armor velocity and wave height NOT exceeded	Default Adjustment Factor: Critical armor velocity and/or wave height exceeded	User-Specified Adjustment Factor: Critical armor velocity and wave height NOT exceeded	User-Specified Adjustment Factor: Critical armor velocity and/or wave height exceeded
A2 - Armor not present, Levee	aL_2_NE, aL_2_E	1	1		
A2 - Armor not present, Foundation	aF_2_NE, aF_2_E	1	1		

	Critical velocity at which armor fails (ft/s), CVL_a_L and CVL_a_F	Critical wave height at which armor fails (ft), Wcr_a_L and Wcr_a_F
... for levee	2	2
... for foundation	2	2

4. Select Vegetation Type and Review/Edit Adjustment Factor for Levee Erosion Rate

Vegetation Type	Symbol	Default Adjustment Factor: Critical vegetation velocity and wave height NOT exceeded	Default Adjustment Factor: Critical vegetation velocity and/or wave height exceeded	User-Specified Adjustment Factor: Critical vegetation velocity and wave height NOT exceeded	User-Specified Adjustment Factor: Critical vegetation velocity and/or wave height exceeded
V3 - Neutral or None	aL_3_NE, aL_3_E	1	1		

Critical velocity at which veg. protection is lost (ft/s), CVL_v	Critical wave height at which veg. protection is lost (ft), Wcr_v
1	1

5. Select Levee Soil Type and Review/Edit Statistical Parameters of Erosion Soil Properties

		Default Values				User-Specified Values			
		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)	
Levee Soil Type	Symbol	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation
LSO5 - Very Erodible	τ_{CL}, k_L	0.003	0.785	1.867	0.473				

6. Select Foundation Soil Type and Review/Edit Statistical Parameters of Erosion Soil Properties

		Default Values				User-Specified Values			
		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)	
Foundation Soil Type	Symbol	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation
FSO5 - Very Erodible	τ_{CF}, k_F	0.003	0.785	1.867	0.473				

7. Provide Information on Relevant Channel, Levee, and Foundation Attributes

Effective levee width against erosion (ft)	W_E	10
Levee slope (X Horizontal to 1 Vertical; Specify X)	X_L	2
Foundation slope (X Horizontal to 1 Vertical; Specify X)	X_F	2
Landside toe elevation, NAVD 88 (ft)	LTE	4854
Maximum water surface elevation, NAVD 88 (ft)	LCE	4860
Channel bottom elevation, NAVD 88 (ft)	ZB	4848
Channel bottom width (ft)	B_W	450
Bed roughness (ft)	k_b	0.0008
Levee is on a channel bend? (Yes or No), Bend If Yes, provide the following information.		No
- Radius of the bend (ft)	C_r	10000
- Angle of the bend (degrees)	α	10

Levee Slope Category	Symbol	Default or User-Specified?	Default Adj. Factor	User-Specified Adj. Factor
LS2 - Steep (steeper than 2.5H to 1V)	aL_4	Default	1.2	

Foundation Slope Category	Symbol	Default or User-Specified?	Default Adj. Factor	User-Specified Adj. Factor
FS2 - Steep (steeper than 2.5H to 1V)	aF_1	Default	1.2	

8. Characterize Vulnerability to Wind/Wave Erosion

Is the levee location vulnerable to wind/wave erosion impact?	Wind	No
If the answer is No, skip the rest of this step. If the answer is Yes, provide the following information:		
Levee slope roughness (ft)	k_l	0.0003

Maximum wind speed against the levee face (miles/hour)	Mean	Coefficient of Variation	Upperbound, Upper_U_pi
	50	0.5	80

Maximum fetch length (ft)	F	600
Duration of Wind (hrs)	d_w	2
Efficiency of wave breaking to erode sediment	Eff	7.50%

9. Current Speed

	Water Surface Elevation, NAVD 88 (ft)	Velocity (ft/sec)	Duration, d (hours)	Velocity based on Mannings coeff. (ft/sec)
	4855	3.6	0	5.4
	4856.5	3.2	84	6.2
	4859	3.5	0	7.2
TOL	4860	3.5	0	7.7

Optional Input
Mannings coefficient equivalent velocity calculator
(Note: For information only, not needed for computations)

Channel bottom slope (X Hori. to 1 Vert.; Specify X), X_CB	600
Mannings coefficient for channel bottom, n	0.04

Table C-14 Station 515000 Erosion Output 50-year event (average water elevation = 4856.5 feet)

Levee ID	Winslow Levee		
Location	Station 51500		
Beginning	Station 51500	**Ending**	Station 51500
Notes	Trial 1 -Partially Engineered/cutoffed Levee, No Armor, No Vegetation, Very Erodible Levee and Foundation, No Wind Waves, 10 to 100 yr floods		

Run Time: *3/23/2012 13:13*

				Levee Erosion (ft)		Foundation Erosion (ft)	
Water Surface Elevation, NAVD 88 (ft)	**Probability of Levee Erosion Failure**	**Probability of Foundation Erosion Failure**	**Total Probability of Erosion Failure**	**Mean**	**Standard Deviation**	**Mean**	**Standard Deviation**
4855	0	0	0.000	0.000	0.000	0.000	0.000
4856.5	0.008	0.005	0.013	3.803	1.808	3.765	1.717

Table C-15 Station 515000 Erosion inputs 50-year event (average water elevation = 4857 feet)

1. Provide Project ID information

Levee ID	Winslow Levee		
Location	Station 51500		
Beginning Location (levee miles, river miles or stations)	Station 51500	Ending Location	Station 51500
Notes	Trial 1 -Partially Engineered/cutoffed Levee, No Armor, No Vegetation, Very Erodible Levee and Foundation, No Wind Waves, 10 to 100 yr floods		

2. Select Levee Type and Review/Edit Adjustment Factor for Levee Erosion Rate

Levee Type	Symbol	Default Adjustment Factor	User-Specified Adjustment Factor
L1 - Levees with Cutoffs	aL_1	1	

(Note: L1 levee type includes homogenous levees, levees with internal cutoff walls, zoned/partially engineered levees, and floodwalls type A)

3. Select Armor Category and Review/Edit Adjustment Factor for Levee and Foundation Erosion Rate

		Default Adjustment Factor		User-Specified Adjustment Factor	
Armor Category	Symbol	Critical armor velocity and wave height NOT exceeded	Critical armor velocity and/or wave height exceeded	Critical armor velocity and wave height NOT exceeded	Critical armor velocity and/or wave height exceeded
A2 - Armor not present, Levee	aL_2_NE, aL_2_E	1	1		
A2 - Armor not present, Foundation	aF_2_NE, aF_2_E	1	1		

	Critical velocity at which armor fails (ft/s), CVL_a_L and CVL_a_F	Critical wave height at which armor fails (ft), Wcr_a_L and Wcr_a_F
... for levee	2	2
... for foundation	2	2

4. Select Vegetation Type and Review/Edit Adjustment Factor for Levee Erosion Rate

		Default Adjustment Factor		User-Specified Adjustment Factor	
Vegetation Type	Symbol	Critical vegetation velocity and wave height NOT exceeded	Critical vegetation velocity and/or wave height exceeded	Critical vegetation velocity and wave height NOT exceeded	Critical vegetation velocity and/or wave height exceeded
V3 - Neutral or None	aL_3_NE, aL_3_E	1	1		

Critical velocity at which veg. protection is lost (ft/s), CVL_v	Critical wave height at which veg. protection is lost (ft), Wcr_v
1	1

5. Select Levee Soil Type and Review/Edit Statistical Parameters of Erosion Soil Properties

		Default Values				User-Specified Values			
		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)	
Levee Soil Type	Symbol	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation
LSO5 - Very Erodible	τ_{cL}, k_L	0.003	0.785	1.867	0.473				

6. Select Foundation Soil Type and Review/Edit Statistical Parameters of Erosion Soil Properties

		Default Values				User-Specified Values			
		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)		Critical Shear Stress (psf)		Erodibility Coefficient (ft^3/lb-hr)	
Foundation Soil Type	Symbol	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation	Mean	Coefficient of Variation
FSO5 - Very Erodible	τ_{cF}, k_F	0.003	0.785	1.867	0.473				

7. Provide Information on Relevant Channel, Levee, and Foundation Attributes

Effective levee width against erosion (ft)	W_E	10
Levee slope (X Horizontal to 1 Vertical; Specify X)	X_L	2
Foundation slope (X Horizontal to 1 Vertical; Specify X)	X_F	2
Landside toe elevation, NAVD 88 (ft)	LTE	4854
Maximum water surface elevation, NAVD 88 (ft)	LCE	4860
Channel bottom elevation, NAVD 88 (ft)	ZB	4848
Channel bottom width (ft)	B_w	450
Bed roughness (ft)	k_b	0.0008
Levee is on a channel bend? (Yes or No), Bend If Yes, provide the following information.		No
- Radius of the bend (ft)	C_r	10000
- Angle of the bend (degrees)	α	10

Levee Slope Category	Symbol	Default or User-Specified?	Default Adj. Factor	User-Specified Adj. Factor
LS2 - Steep (steeper than 2.5H to 1V)	aL_4	Default	1.2	

Foundation Slope Category	Symbol	Default or User-Specified?	Default Adj. Factor	User-Specified Adj. Factor
FS2 - Steep (steeper than 2.5H to 1V)	aF_1	Default	1.2	

8. Characterize Vulnerability to Wind/Wave Erosion

Is the levee location vulnerable to wind/wave erosion impact?	Wind	No
If the answer is No, skip the rest of this step. If the answer is Yes, provide the following information:		
Levee slope roughness (ft)	k_l	0.0003

	Mean	Coefficient of Variation	Upperbound, Upper_U_pi
Maximum wind speed against the levee face (miles/hour)	50	0.3	30
Maximum fetch length (ft)	F		600
Duration of Wind (hrs)	d_w		2
Efficiency of wave breaking to erode sediment	Eff		7.50%

9. Current Speed

	Water Surface Elevation, NAVD 88 (ft)	Velocity (ft/sec)	Duration, d (hours)	Velocity based on Mannings coeff. (ft/sec)
	4855	3.6	0	5.4
	4857	3.3	84	6.4
	4859	3.5	0	7.2
TOL	4860	3.5	0	7.7

Optional Input
Mannings coefficient equivalent velocity calculator
(Note: For information only, not needed for computations)

Channel bottom slope (X Hori. to 1 Vert.; Specify X), X_CB	600
Mannings coefficient for channel bottom, n	0.04

Table C-16 Station 515000 Erosion Output 50-year event (average water elevation = 4857 feet)

Levee ID	Winslow Levee		
Location	Station 51500		
Beginning	Station 51500	**Ending**	Station 51500
Notes	Trial 1 -Partially Engineered/cutoffed Levee, No Armor, No Vegetation, Very Erodible Levee and Foundation, No Wind Waves, 10 to 100 yr floods		

Run Time: *3/23/2012 13:20*

				Levee Erosion (ft)		Foundation Erosion (ft)	
Water Surface Elevation, NAVD 88 (ft)	**Probability of Levee Erosion Failure**	**Probability of Foundation Erosion Failure**	**Total Probability of Erosion Failure**	**Mean**	**Standard Deviation**	**Mean**	**Standard Deviation**
4855	0	0	0.000	0.000	0.000	0.000	0.000
4857	0.01	0.013	0.023	4.138	1.990	4.103	2.098

Appendix D: Overtopping Erosion Calculation Summaries

Overtopping erosion was calculated by calculating the force of different levels of water on an inclined plane of 1.5:1 to 3:.0 to 1 (H:V) that were parallel to the slope. The critical shear stress of the soil was estimated from grain size of the soil or rip-rap, and then the applied shear stress was compared to the critical shear stress and an erosion rate was established. As shown in Figure 14 of the text, the duration of storm peaks is very short. Therefore it was judged that 1 hour of sustained overtopping would be an appropriate time to integrate the erosion rate to determine total erosion. For Stations 37000 and 29000 it was estimated that if the levee was overtopped by 2 feet erosion through the rip-rap may occur, which would quickly erode the levee soils and cause unsatisfactory performance. A P_u=0.5 was therefore assigned at 2 feet of overtopping. Any higher levels of overtopping very quickly result in unsatisfactory performance.

Applied overtopping shear Stress (psf)

Slope	Angle (rad)	Angle (degrees)	Shear Stress (H=1ft)	Shear Stress (H=2 ft)	Shear Stress (H=3	Pascals		
1.5:1	0.5880026	33.70715562	34.61329224	69.22658449	103.8398767	1730.7	3461.3	5192
2.0:1	0.4636476	26.57852536	27.90612836	55.81225672	83.71838508	1395.3	2790.6	4185.9
2.5:1	0.3805064	21.81246748	23.1747782	46.34955641	69.52433461	1158.7	2317.5	3476.2
3.0:1	0.3217506	18.4442993	19.7326126	39.4652252	59.1978378	986.63	1973.3	2959.9

Section 495 and 51500

Estimated Critical Shear stress				Overtopped by 1 foot	
Pascals	psf	Erosion Rate (1ft overtopping) ft/hr	erodibility coefficient(ft^3/lb-hr)	Erosion (ft) in 1 hr	
1	0.02	34.59329224	1.00E+00	34.59329224	Pu=100 % if overtops for 1 hr

Section 37000 and 29000

Estimated Critical Shear Stres		Erosion Rate (1ft overtopping) ft/hr	erodibility coefficient(ft^3/lb-hr)	Erosion (ft) in 1 hr
Pascals	psf			
318	6.36	0.282532922	1.00E-02	0.282532922
		Erosion Rate (2ft overtopping) ft/hr	erodibility coefficient(ft^3/lb-hr)	Erosion (ft) in 1 hr
		6.92E-01	1.00E-02	0.692265845

Rip rap 1 (if erosion makes it through rip rap than breach)

Erosion Calculation Proabilitities

2 feet over top

	Applied stress(psf)	Critical shear Stress (p:	Erodibility coefficien	Estimated Erosion (ft in 1 hı	Criticl Er	FS	
Expecte	69	6.36	0.01	0.63	1	1.5964	
	69	4	0.01	0.65	1	1.5385	
	69	8	0.01	0.61	1	1.6393	0.0026
Expecte	69	6.36	0.01	0.63	1	1.5964	
	69	6.36	0.05	3.13	1	0.3193	
	69	6.36	0.003	0.19	1	5.3214	6.7547
						variance=	6.7572
						stdev=	2.5995
						Expectec	1.5964
						V=	1.6283
						sigmalnF	1.138
						E(lnFs)	-0.052
						beta	-0.046
						Fs critica	1
						Z	0.0461
						Pu	0.5184

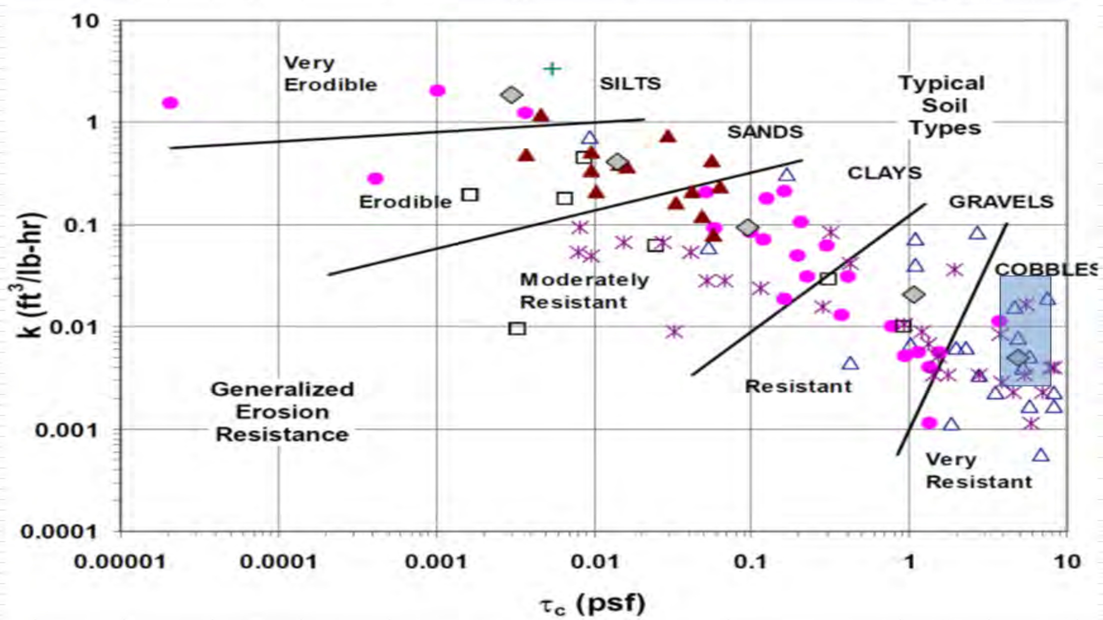


Table D-1 Overtopping Erosion Estimates

ATTACHMENT 5

HYDRAULIC ANALYSIS OF CONVEYANCE MEASURES MEMORANDUM

LITTLE COLORADO RIVER AT WINSLOW
HYDRAULIC AND SEDIMENTATION APPENDIX
APRIL 2016

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CESPL-ED-HH 17 January 2013

MEMORANDUM FOR RECORD

SUBJECT: Little Colorado River at Winslow Feasibility Study – Hydraulic Analysis of Conveyance Measures

1. References
    a. US Army Corps of Engineers, Los Angeles District. *Hydraulics and Sedimentation Appendix*, *Little Colorado River at Winslow Feasibility Study*, March 2012
    b. Navajo County Flood Control District, *Technical Data Notebook for Little Colorado River near Winslow Floodplain Delineation Study*, July 2009.
    c. US Army Corps of Engineers, Los Angeles District. Hydrology and Hydraulics Branch, *Debris Loading on Bridges and Culverts*, August 2004.

2. The purpose of this memorandum is to transmit Hydraulics Section's hydraulic analysis of measures to improve conveyance of the Little Colorado River (LCR) at the BNSF Railroad Bridge for the Little Colorado River at Winslow Feasibility Study. The memorandum describes the additional hydraulic analysis that was completed regarding improving conveyance at the BNSF Railroad Bridge. Specifically, several measures were considered and modeled using Hydrologic Engineering Center River Analysis System (HEC-RAS) version 4.2 beta. The measures were run with the purpose of increasing conveyance at the BNSF Railroad Bridge in an attempt to decrease the flood hazards caused by the overtopping of the bridge as well as the Winslow Levee on the west bank of the LCR. Also note that while some of the measures presented below meet this requirement, the Winslow Levee may not meet freeboard requirements.

Introduction

3. **Introduction to Measures:** The additional hydraulic analyses considered measures that would decrease the water surface elevation (WSE) at the BNSF Railroad Bridge in an attempt to prevent overtopping of the railroad bridge and the Winslow Levee. The measures include excavating and widening the channel, removing saltcedar, lining a portion of the river bottom with concrete, extending the railroad bridge opening, and installing culverts on either side of the railroad bridge. HEC-RAS was used to analyze the various measures, and each measure includes saltcedar removal in some capacity. Extending the bridge opening and installing culverts were two measures that were considered but not modeled as part of this analysis. The analysis was based on the baseline condition 1-percent ACE flood (69,200 cfs).

4. **Reach Description:** The reach of the LCR studied for this analysis begins approximately 1 mile upstream from the BNSF Railroad at station 575+00 and extends approximately 3,500 feet downstream from the I-40 Westbound Bridge to station 470+00. This reach includes four bridges: the BNSF Railroad Bridge (station 529+39.3), the Route 66 / State Route 87 Bridge (524+13.7), and Interstate 40 (I-40) Eastbound (505+88.4) and Westbound Bridges (504+71.6).

Upstream from the BNSF Railroad bridge the river flows in a westerly direction before making a 90 degree turn to the north approximately 1,500 feet upstream from the BNSF Railroad Bridge. Saltcedar is prevalent on both the left and right banks in this area, and the LCR is approximately 100-200 feet wide from station 575+00 to the BNSF Bridge. Downstream of the BNSF Bridge to the I-40 Bridges, the channel is approximately 200 feet wide with saltcedar prevalent on the left and right banks. Downstream from the I-40 Bridges to station 470+00, the channel varies from 150 to 200 feet wide, with saltcedar on the left bank and scattered brush and heavy weeds on the right bank. See Enclosure 4-1 for the Study Area Map.

Modeling Considerations

5. **Manning's Roughness Coefficients:** Manning's Roughness coefficients (n-values) for the main channel and overbanks were estimated based on topographic mapping, aerial photos, as-built drawings, and field investigations. The n-values used in the analysis were based on the Arizona Department of Water Resources *Design Manual for Engineering Analysis of Fluvial Systems*. The n-values used in the model include: 0.03 for the main channel bed consisting of fine to medium sand, 0.05 for floodplains with scattered brush and heavy weeds, 0.085 for areas with medium to dense brush, 0.090 for residential medium density area, and 0.12 for dense willows, heavy timber and saltcedar.

6. **Removal of Saltcedar:** Each measure includes the removal of saltcedar. Enclosures 4-2 to 4-7 show the saltcedar removal areas for each measure. The Manning's n-value for areas with saltcedar was modeled using 0.12. In removing the saltcedar for each measure, the following two assumptions were made. First, a Manning's n-value of 0.05 (scattered brush and heavy weeds) was used for floodplain areas where saltcedar was removed. Second, a Manning's n-value of 0.03 (channel bed, medium to fine sand) was used for saltcedar areas that were excavated to be channel bottom. Using a higher Manning's n-value for the floodplain areas than the channel bottom is a conservative assumption assuming some vegetation re-growth in the floodplain areas along the channel banks.

7. **Bridge Assumptions:** As-built data from Navajo County Flood Control District July 2009 Floodplain Delineation Study was used in the bridge modeling. Deck elevation data was modeled using the 2-foot contour topography as well as the bridge As-built drawings. Pier debris was assumed on the I-40 Bridges. Pier debris was not assumed on the BNSF Railroad Bridge or the Route 66 Bridge, which have pier widths of 7 and 6 feet, respectively. The Los Angeles District Hydrology and Hydraulics Policy Memorandum No. 4 *Debris Loading on Bridges and Culverts* states that "debris loading assumptions are not applicable to circular, elliptical, or streamlined piers above a (width of 6 feet)." Additionally, the four bridges were modeled using pressure and/or weir flow as the modeling high flow method.

8. **BNSF Railroad Pertinent Data**: The Bridge soffit according to the As-built drawings is 4862.68 feet. The bridge deck is 3.75 feet thick, with an approximate elevation of 4866.3 ft. The bridge has a solid steel railing totaling 10 feet high from the bridge soffit to the top of the railing.

Sediment Modeling Considerations

9. **Sediment Analysis:** Preliminary sediment analysis was done for each measure described below. The purpose of running a preliminary sediment analysis on the measures was to see if there were any major problems caused by aggradation or degradation. HEC-RAS 4.2 beta was used to run the sediment models. The 1-percent ACE 3.5 day flood hydrograph was used to complete the preliminary sediment analysis. This provides one example of an extreme event sediment condition and does not represent the regular annual sediment condition. Detailed sedimentation analysis would require eight flood frequency simulations as well as integration calculation for the annual sediment condition.

Existing Model Hydraulic Conditions at BNSF Railroad Bridge

10. For the existing conditions hydraulic model, the 1-percent ACE flood event weir flows over the BNSF Bridge with an upstream WSE of 4867.67 feet. The railroad on the left side of the bridge has an elevation of approximately 4866.6 feet according to the 2-foot contour topography. Additionally, the Winslow Levee on the upstream side of the BNSF Bridge has an elevation of approximately 4865.8 feet at station 529+83.27. The Winslow Levee has a low spot approximately 250 feet upstream from the bridge consisting of an elevation of about 4865 ft. Consequently, for the 1-percent ACE flood event, the LCR not only overtops the BNSF Railroad Bridge but also the Winslow Levee, allowing flows to travel towards the City of Winslow. The flows approaching the BNSF Railroad have a velocity varying from 2.5 to 4.1 ft/sec. After weir flowing over the BNSF Bridge, the flows increase velocity to approximately 13 ft/sec under the Route 66 Bridge.

Measure A: Excavate and Widen Channel (Station 570+00 to 470+00)

11. **Description:** The first measure includes the excavation and widening of the channel bottom from approximately 4,000 feet upstream from the BNSF Railroad Bridge at station 570+00 to approximately 3,500 feet downstream from the I-40 Bridges at station 470+00. See Enclosure 4-2 for a map of Measure A. Additionally, no attempt was made to deepen the channel throughout the reach. Measure A includes the removal of approximately 60 acres of saltcedar throughout the reach. The excavation and widening of the channel begins at station 570+00. The excavated channel varies from 150 feet at station 570+00 to 650 feet at the BNSF Railroad Bridge. Downstream from the BNSF Bridge, the channel widens to approximately 850 feet near the I-40 Bridges. Downstream from the I-40 Bridges, the channel width varies from 850 feet at the I-40 Bridges to 130 feet at station 470+00. This measure includes approximately 10,000 feet of LCR channel excavation and widening.

12. **Hydraulic Analysis Results:** The 1-percent ACE flood event for this measure does not overtop the BNSF Bridge nor does it overtop the Winslow Levee upstream from the BNSF Bridge. The increased capacity at the BNSF Bridge results in a pressure flow situation. The 1-percent ACE flood event WSE at the upstream end of the BNSF Railroad at station 529+83.27 is 4863.27. The flows approaching the BNSF Railroad have a velocity varying from 4.5 to 6.7 ft/sec. After pressure flowing under the BNSF Bridge, the velocity varies from 6 to 7 ft/sec as flows travel under the Route 66 Bridge.

13. **Sediment Analysis Results:** Using the 1-percent ACE flood event 3.5 day hydrograph, the sediment analysis simulation for Conveyance Measure A resulted in aggradation and degradation in the study reach without overtopping the BNSF Bridge or the Winslow Levee. Upstream from the BNSF Railroad from station 560+00 to 535+00 the streambed experiences aggradation of as much as 1.8 feet. From the BNSF Bridge to the Route 66 Bridge (station 535+00 to 524+93), the streambed degrades as much 0.5 feet. From the Route 66 Bridge to the I-40 Eastbound Bridge (station 524+93 to 506+75.8), the channel aggrades as much as 1.5 feet. Under the I-40 Bridges from station 506+75.8 to 503+69, the channel degrades as much as 2.5 feet. Downstream from the I-40 Bridges until station 470+00, the streambed aggrades as much as 0.3 feet.

Measure B: Excavate and Widen Channel (Station 540+00 to 480+00)

14. **Description:** The second measure includes the excavation and widening of the channel bottom from approximately 1,000 feet upstream from the BNSF Railroad Bridge at station 540+00 to approximately 2,500 feet downstream from the I-40 Westbound Bridge at station 480+00. This measure includes approximately 6,000 feet of excavation and widening of the LCR. No attempt was made to deepen the channel throughout the reach. See Enclosure 4-3 for a map of Measure B. This measure includes the removal of approximately 85 acres of saltcedar throughout the reach. The excavation and widening of the channel begins at station 540+00. The excavated channel varies from 200 feet at station 540+00 to 650 feet at the BNSF Railroad Bridge (529+39.3). Downstream from the BNSF Bridge, the channel maintains a width of approximately 600 feet from the Route 66 to I-40 Bridges. Downstream from the I-40 Bridges, the channel width varies from 600 feet the I-40 Bridges to 150 feet at station 480+00.

15. Analysis Results: The 1-percent ACE flood event for this measure does not overtop the BNSF Bridge nor does it overtop the Winslow Levee upstream from the BNSF Bridge. The increased capacity at the BNSF Bridge results in a pressure flow situation. The 1-percent ACE flood event WSE at the upstream end of the BNSF Railroad at station 529+83.27 is 4863.85. The bridge soffit is at an elevation of 4862.68 ft, so this measure results in approximately 1 ft of water impacting the bridge as it pressure flows under. The flows approaching the BNSF Railroad have a velocity varying from 3.8 to 8.4 ft/sec. After pressure flowing under the BNSF Bridge, the velocity is approximately 7 ft/sec flowing beneath the Route 66 Bridge.

16. **Sediment Analysis Results:** Using the 1-percent ACE flood event 3.5 day hydrograph, the sediment analysis simulation for Measure B resulted in aggradation and degradation in the study reach without overtopping the BNSF Bridge or the Winslow Levee. Upstream from the BNSF Bridge from station 570+00 to 535+00 the streambed experiences degradation of as much as 3.6 feet. From the BNSF Railroad Bridge (station 535+00) to the I-40 Eastbound Bridge (station 506+75.8), the streambed aggrades as much 2.1 feet. Under the I-40 Bridges from station 506+75.8 to 503+69, the channel degrades as much as 3.6 feet. Downstream from the I-40 Bridges until station 480+00, the streambed aggrades as much as 0.2 feet.

Measure C: Excavate and Widen Channel (Station 540+00 to 515+00)

17. **Description:** The third measure includes the excavation and widening of the channel bottom from approximately 1,000 feet upstream from the BNSF Railroad Bridge at station 540+00 to approximately 1,000 feet downstream from the Route 66 Bridge at station 515+00. This measure includes approximately 2,500 feet of excavation and widening of the LCR. No attempt was made to deepen the channel throughout the reach. See Enclosure 4-4 for a map of Measure C. This measure includes the removal of approximately 96 acres of saltcedar throughout the reach. The excavation and widening of the channel begins at station 540+00. The excavated channel varies from 200 feet at station 540+00 to 650 feet at the BNSF Railroad Bridge (529+39.3). Downstream from the BNSF Railroad Bridge, the channel maintains a width of approximately 650 feet through the Route 66 Bridge. Downstream from the Route 66 Bridge, the channel width varies from 650 feet to 200 feet at station 515+00.

18. **Analysis Results:** The 1-percent ACE flood event for this measure does not overtop the BNSF Bridge nor does it overtop the Winslow Levee upstream from the BNSF Bridge. The upstream flows do not reach the low point in the Winslow Levee (4865 ft); however, the freeboard is less than 0.5 ft approximately 250 feet upstream from the BNSF Railroad Bridge. The increased capacity at the BNSF Bridge results in a pressure flow situation. The 1-percent ACE flood event WSE at the upstream end of the BNSF Railroad at station 529+83.27 is 4864.76. The bridge soffit is at an elevation of 4862.68 ft, so this measure results in approximately 2 ft of water impacting the bridge as it pressure flows under. The flows approaching the BNSF Railroad have a velocity varying from 2.8 to 8.0 ft/sec. After pressure flowing under the BNSF Bridge, the velocity is approximately 7 ft/sec beneath the Route 66 Bridge.

19. **Sediment Analysis Results:** Using the 1-percent ACE flood event 3.5 day hydrograph, the sediment analysis simulation for Measure C resulted in aggradation and degradation in the study reach without overtopping the BNSF Bridge or the Winslow Levee. Upstream from the BNSF Bridge from station 550+00 to 535+00 the streambed experiences degradation of as much as 1.0 feet. From the BNSF Railroad Bridge (station 535+00) to the I-40 Eastbound Bridge (station

506+75.8), the streambed aggrades as much 0.9 feet. Under the I-40 Bridges from station 506+75.8 to 503+69, the channel degrades as much as 5.3 feet. Downstream from the I-40 Bridges until station 480+00, the streambed aggrades as much as 0.3 feet.

Measure D: Remove Saltcedar (Tamarisk)

20. **Description:** The fourth measure includes the removal of saltcedar from the existing channel banks beginning approximately 4,500 feet upstream from the BNSF Railroad Bridge at station 575+00 to the I-40 Bridges (station 503+69). See Enclosure 4-5 for a map of Measure D and the areas where saltcedar was removed. This measure includes the removal of approximately 110 acres of saltcedar throughout the reach. This measure covers approximately 7,500 feet of the LCR.

21. **Analysis Results:** The 1-percent ACE flood event for this measure does overtop the BNSF Bridge as well as the Winslow Levee upstream from the BNSF Bridge. The removal of saltcedar alone in this reach does not prevent the 1-percent ACE flood event from overtopping the BNSF Bridge with an upstream WSE of 4867.26 ft. The LCR weir flows around the BNSF Bridge, causing the Winslow Levee to be overtopped and flood waters to reach the City of Winslow. The bridge soffit is at an elevation of 4862.68 ft, so this measure results in approximately 4.5 ft of water impacting the upstream side of the bridge as it weir flows around it. The flows approaching the BNSF Railroad have a velocity varying from 2.3 to 4.1 ft/sec. After weir flowing over the BNSF Bridge, the flows increase velocity to approximately 10.5 ft/sec under the Route 66 Bridge.

22. **Sediment Analysis Results:** Using the 1-percent ACE flood event 3.5 day hydrograph, the sediment analysis simulation for Measure D resulted in aggradation and degradation in the study reach while overtopping the BNSF Bridge and the Winslow Levee. Upstream from the BNSF Bridge from station 540+00 to the Route 66 Bridge (station 523+28), the streambed degrades as much as 1.8 feet. From the Route 66 Bridge (station 523+28) to the I-40 Eastbound Bridge (station 506+75.8), the streambed aggrades as much 3.0 feet. Under the I-40 Bridges from station 506+75.8 to 503+69, the channel degrades as much as 6.0 feet. Downstream from the I-40 Bridges until station 480+00, the streambed aggrades as much as 0.3 feet.

Measure E: Excavate and Widen Channel – Lined in Concrete (Station 540+00 to 480+00)

23. **Description:** The fifth measure includes the excavation and widening of the channel bottom from approximately 1,000 feet upstream from the BNSF Railroad Bridge at station 540+00 to approximately 2,500 feet downstream from the I-40 Westbound Bridge at station 480+00. This measure is a concrete lined version of Measure B. This measure includes approximately 6,000 feet of concrete-lined channel. No attempt was made to deepen the channel throughout the reach. See Enclosure 4-6 for a map of Measure E. This measure includes the removal of approximately 85 acres of saltcedar throughout the reach. The excavation and widening of the

channel begins at station 540+00. The concrete-lined, excavated channel varies from 200 feet at station 540+00 to 650 feet at the BNSF Railroad Bridge (529+39.3). Downstream from the BNSF Bridge, the channel maintains a width of approximately 600 feet from the Route 66 to I-40 Bridges. Downstream from the I-40 Bridges, the concrete-lined channel width varies from 600 feet to 150 feet at station 480+00.

24. Analysis Results: The 1-percent ACE flood event for this measure does not overtop the BNSF Bridge nor does it overtop the Winslow Levee upstream from the BNSF Bridge. The increased capacity at the BNSF Bridge results in a pressure flow situation. The 1-percent ACE flood event WSE at the upstream end of the BNSF Railroad at station 529+83.27 is 4863.68, which is slightly lower than Measure B. The bridge soffit is at an elevation of 4862.68 ft, so this measure results in approximately 1 ft of water impacting the bridge as it pressure flows under. The flows approaching the BNSF Railroad have a velocity varying from 5.0 to 10.5 ft/sec. After pressure flowing under the BNSF Bridge, the velocity is approximately 7.5 ft/sec flowing beneath the Route 66 and I-40 Bridges.

25. **Sediment Analysis Results:** Using the 1-percent ACE flood event 3.5 day hydrograph, the sediment analysis simulation for Measure E resulted in aggradation and degradation in the study reach without overtopping the BNSF Bridge or the Winslow Levee. Upstream from the BNSF Bridge from station 570+00 to 535+00, the streambed degrades as much as 8 feet. From the BNSF Railroad Bridge to the I-40 Eastbound Bridge (station 506+75.8), the streambed aggrades as much 3.5 feet. From station 506+75.8 to 480+00, the channel aggrades as much as 0.5 feet.

Measure F: Excavate and Widen Channel – Lined in Concrete (Station 540+00 to 515+00)

26. **Description:** The sixth measure includes the excavation and widening of the channel bottom from approximately 1,000 feet upstream from the BNSF Railroad Bridge at station 540+00 to approximately 1,000 feet downstream from the Route 66 Bridge at station 515+00. This measure is a concrete lined version of Measure C. This measure includes approximately 2,500 feet of the concrete-lined channel. No attempt was made to deepen the channel throughout the reach. See Enclosure 4-7 for a map of Measure F. This measure includes the removal of approximately 95 acres of saltcedar throughout the reach. The excavation and widening of the channel begins at station 540+00. The concrete-lined, excavated channel varies from 200 feet at station 540+00 to 650 feet at the BNSF Railroad Bridge (529+39.3). Downstream from the BNSF Bridge, the channel maintains a width of approximately 650 feet through the Route 66 Bridge. Downstream from the Route 66 Bridge, the channel width varies from 650 feet to 200 feet at station 515+00.

27. **Analysis Results:** The 1-percent ACE flood event for this measure does not overtop the BNSF Bridge nor does it overtop the Winslow Levee upstream from the BNSF Bridge. The increased capacity at the BNSF Bridge results in a pressure flow situation. The 1-percent ACE

flood event WSE at the upstream end of the BNSF Railroad at station 529+83.27 is 4864.43, which is slightly lower than Measure C. The bridge soffit is at an elevation of 4862.68 ft, so this measure results in approximately 2 ft of water impacting the bridge as it pressure flows under. The flows approaching the BNSF Railroad have a velocity varying from 6 to 10 ft/sec. After pressure flowing under the BNSF Bridge, the velocity is approximately 7 ft/sec flowing beneath the Route 66 Bridge.

28. **Sediment Analysis Results:** Using the 1-percent ACE flood event 3.5 day hydrograph, the sediment analysis simulation for Measure D resulted in aggradation and degradation in the study reach while overtopping the BNSF Bridge and the Winslow Levee. Upstream from the BNSF Bridge from station 550+00 to 540+00, the streambed degrades as much as 2.0 feet. From station 540+00 to the I-40 Eastbound Bridge (station 506+75.8), the streambed aggrades as much 2.0 feet. Under the I-40 Bridges from station 506+75.8 to 503+69, the channel degrades as much as 5.0 feet. Downstream from the I-40 Bridges until station 480+00, the streambed aggrades as much as 0.3 feet.

Conclusion

29. Hydraulic analysis was performed for six measures: Measures A to C include excavation and widening of the channel bottom for distances of 10,000 feet, 6,000 feet, and 2,500 feet, respectively; Measure D includes the removal of saltcedar; and Measures E and F are concrete-lined measures relating to Measures B and C, respectively. Measures A, B, C, E, and F all meet the goal of getting the 1-percent ACE flood event to not overtop the BNSF Railroad or the Winslow Levee upstream from the bridge. Each conveyance measure has varying water surface elevations at the BNSF Bridge, and each of the five measures are pressure flow under the bridge. Structural analysis would be required to see if the BNSF Railroad Bridge could withstand the pressure applied from the 1-percent ACE flood event for these measures. Measure D involves removing the saltcedar from the existing channel banks. This measure does not successfully prevent the flows from overtopping the BNSF Bridge or the Winslow Levee.

Encls

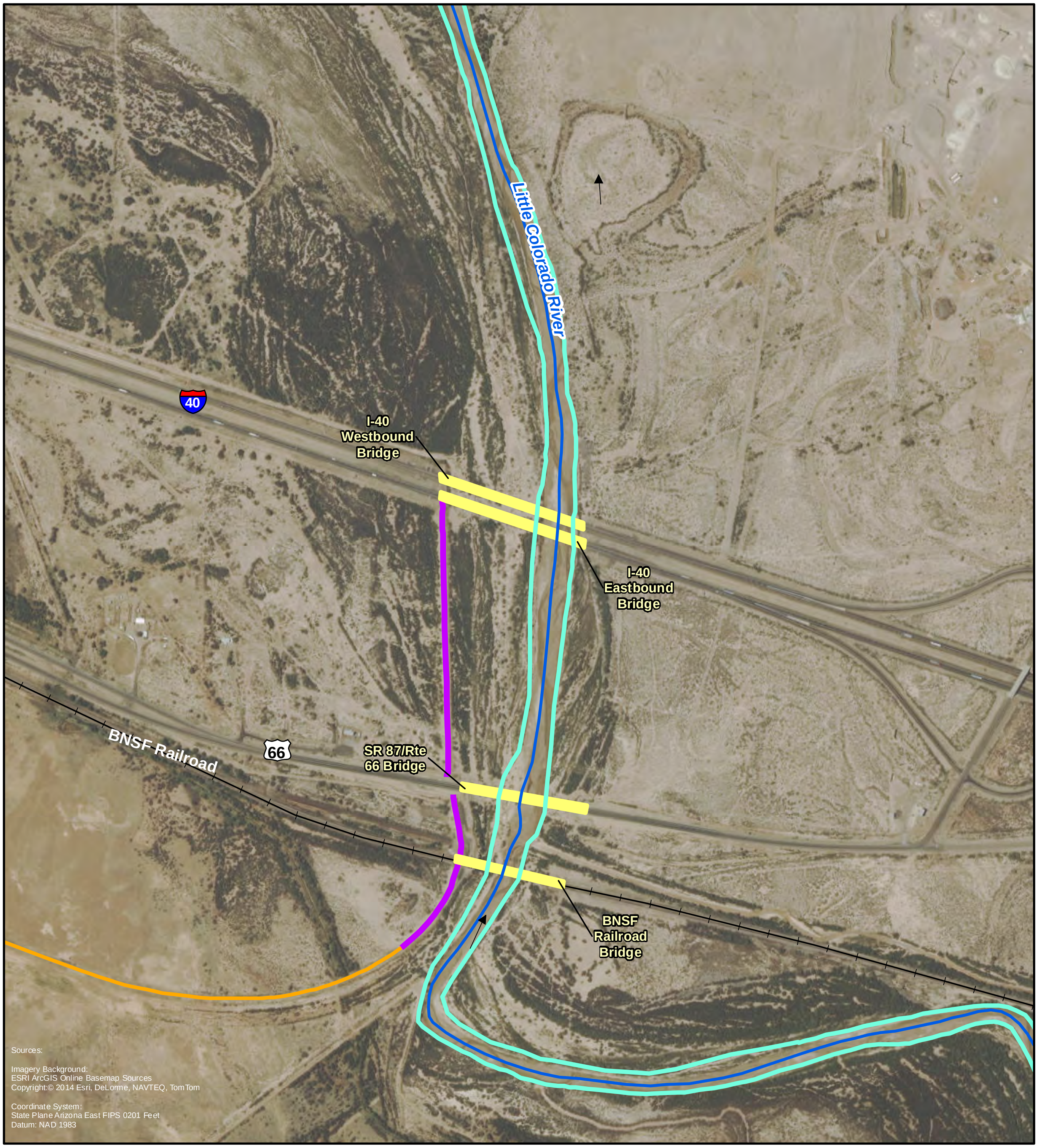
Van Crisostomo, P.E
Chief, Hydraulics Section

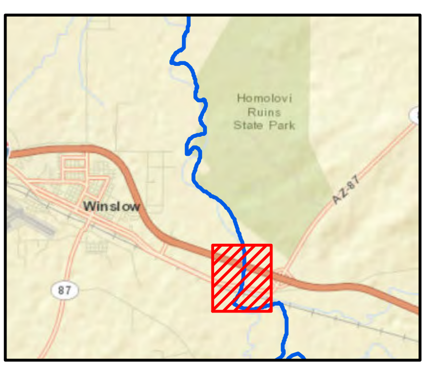
CF: CESPL-PM-C (Kenny)
CESPL-PD-WC (Legere)
CESPL-ED-H
CESPL-ED-HH (2)

CRISOSTOMO
CESPL-ED-HH

CHIEH
CESPL-ED-HH

BIER
CESPL-ED-HH





Legend

- Little Colorado River
- Winslow Levee (Existing)
- Banks (Existing)
- BNSF Railroad
- Bridges
- RWDL (Existing)

N

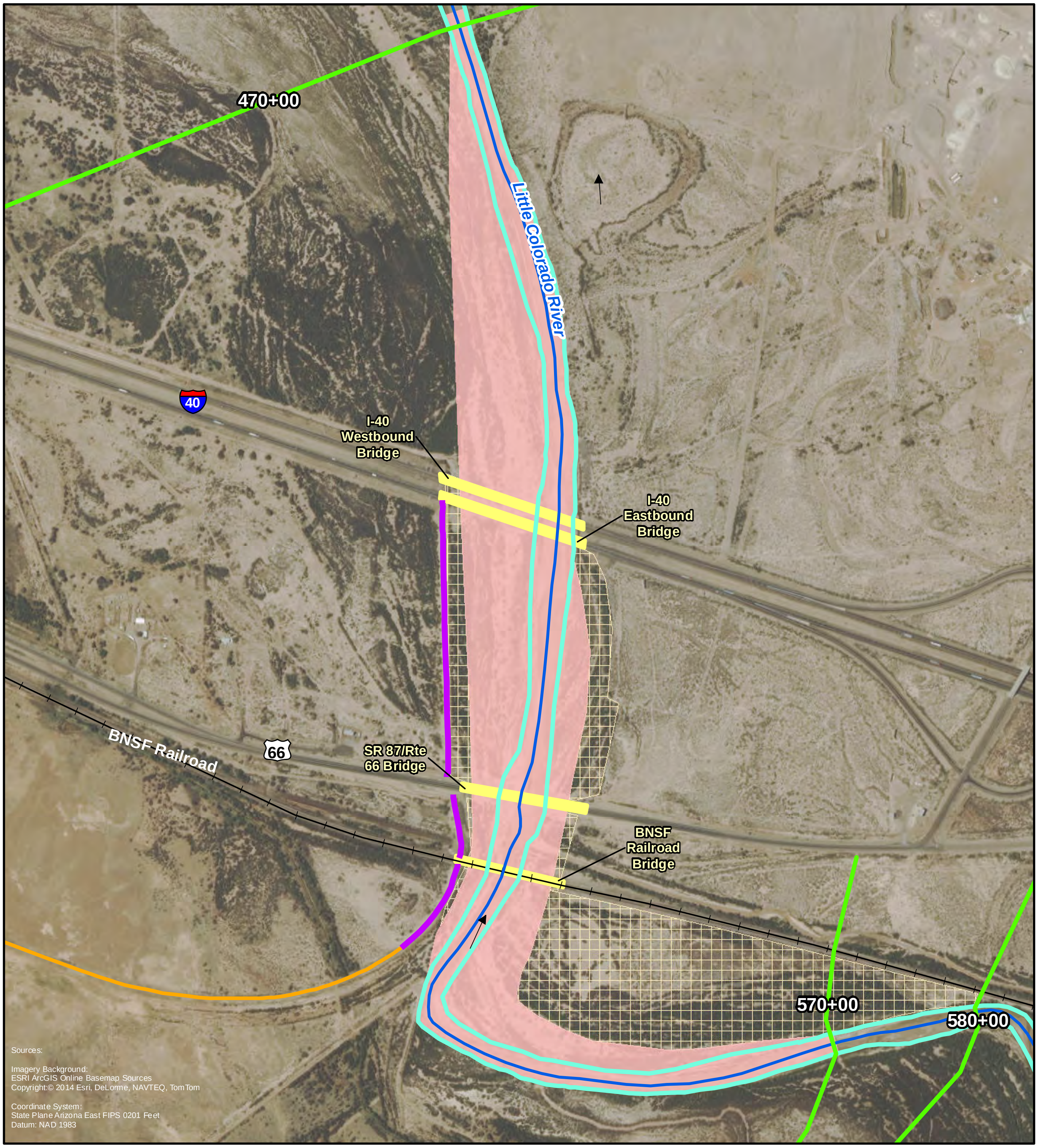
0 500 1,000 2,000 Feet

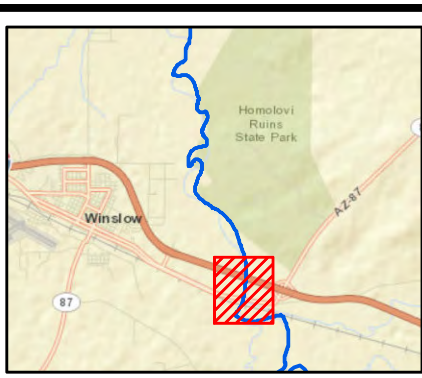
1 in = 1,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

BRIDGE LOCATION MAP

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT





Legend

- Bounding Cross-Sections
- Little Colorado River
- Banks (Existing)
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad
- Bridges
- Excavation Area Measure A
- Remove Saltcedar

N

0 500 1,000 2,000 Feet

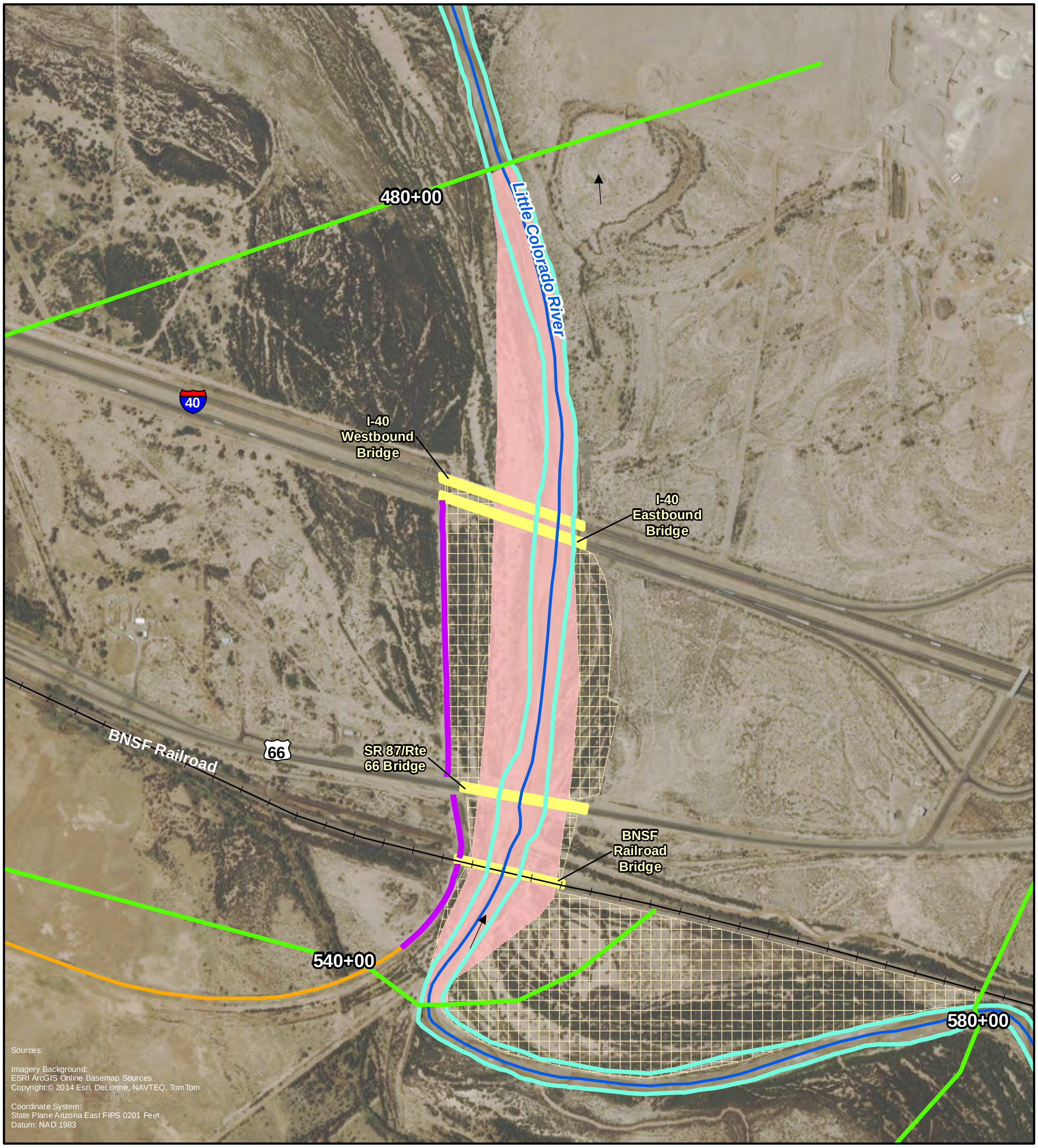
1 in = 1,000 feet

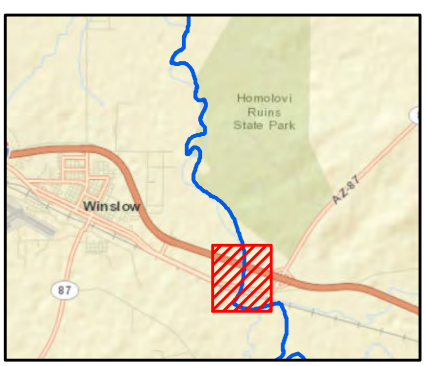
LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

MEASURE A
EXCAVATE & WIDEN CHANNEL
STATION 570+00 TO 470+00
(EARTHEN CHANNEL)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

ENCLOSURE 4-2





Legend

- Bounding Cross-Sections
- Little Colorado River
- Banks (Existing)
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad
- Bridges
- Excavation Area Measure B
- Remove Saltcedar

N

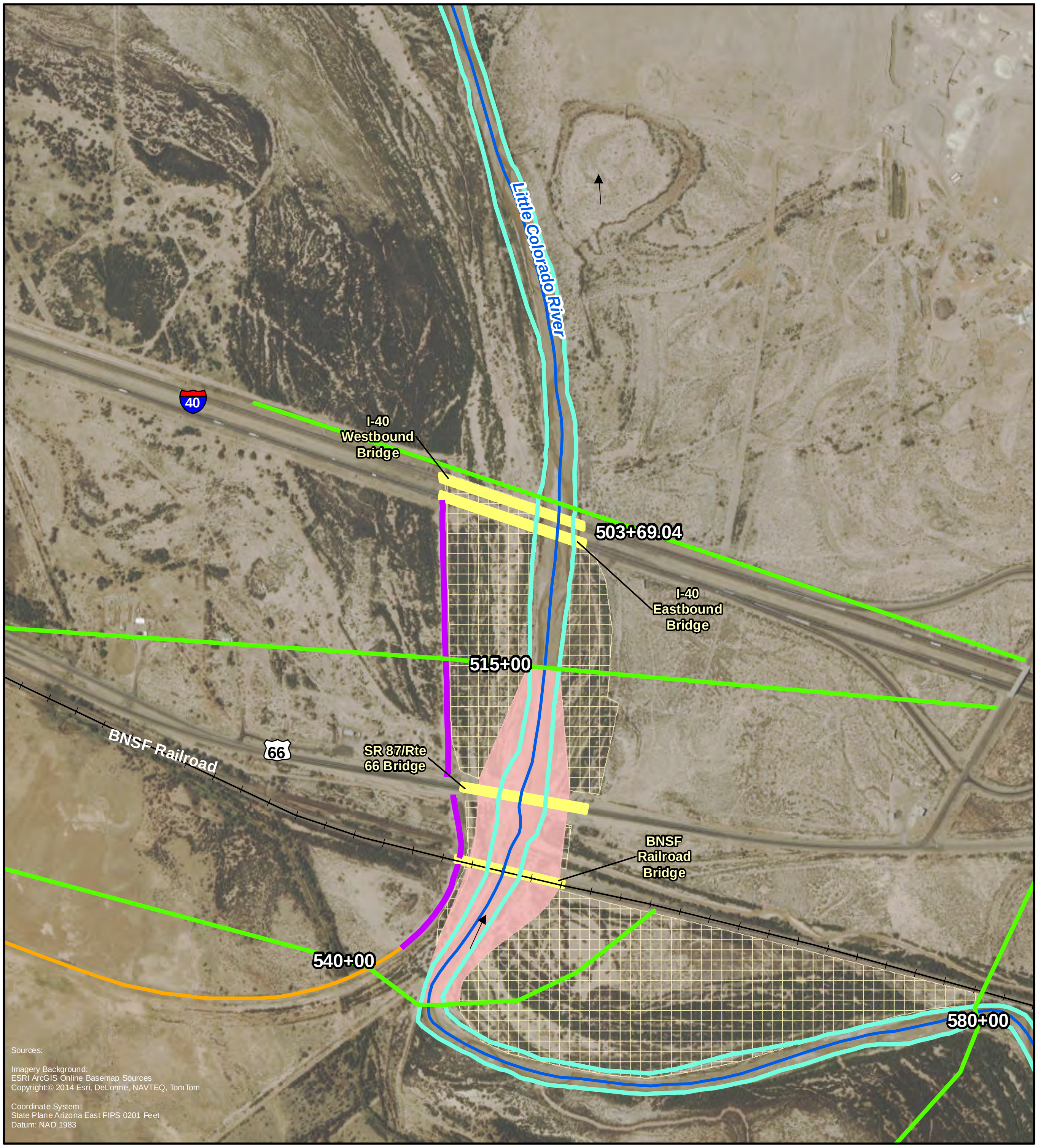
0 500 1,000 2,000 Feet

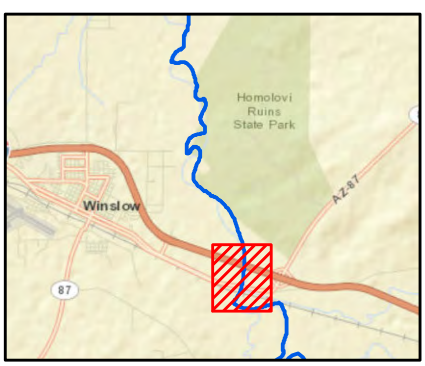
1 in = 1,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

MEASURE B
EXCAVATE & WIDEN CHANNEL
STATION 540+00 TO 480+00
(EATHEN CHANNEL)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT





Legend

- Bounding Cross-Sections
- Little Colorado River
- Banks (Existing)
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad
- Bridges
- Excavation Area Measure C
- Remove Saltcedar

N

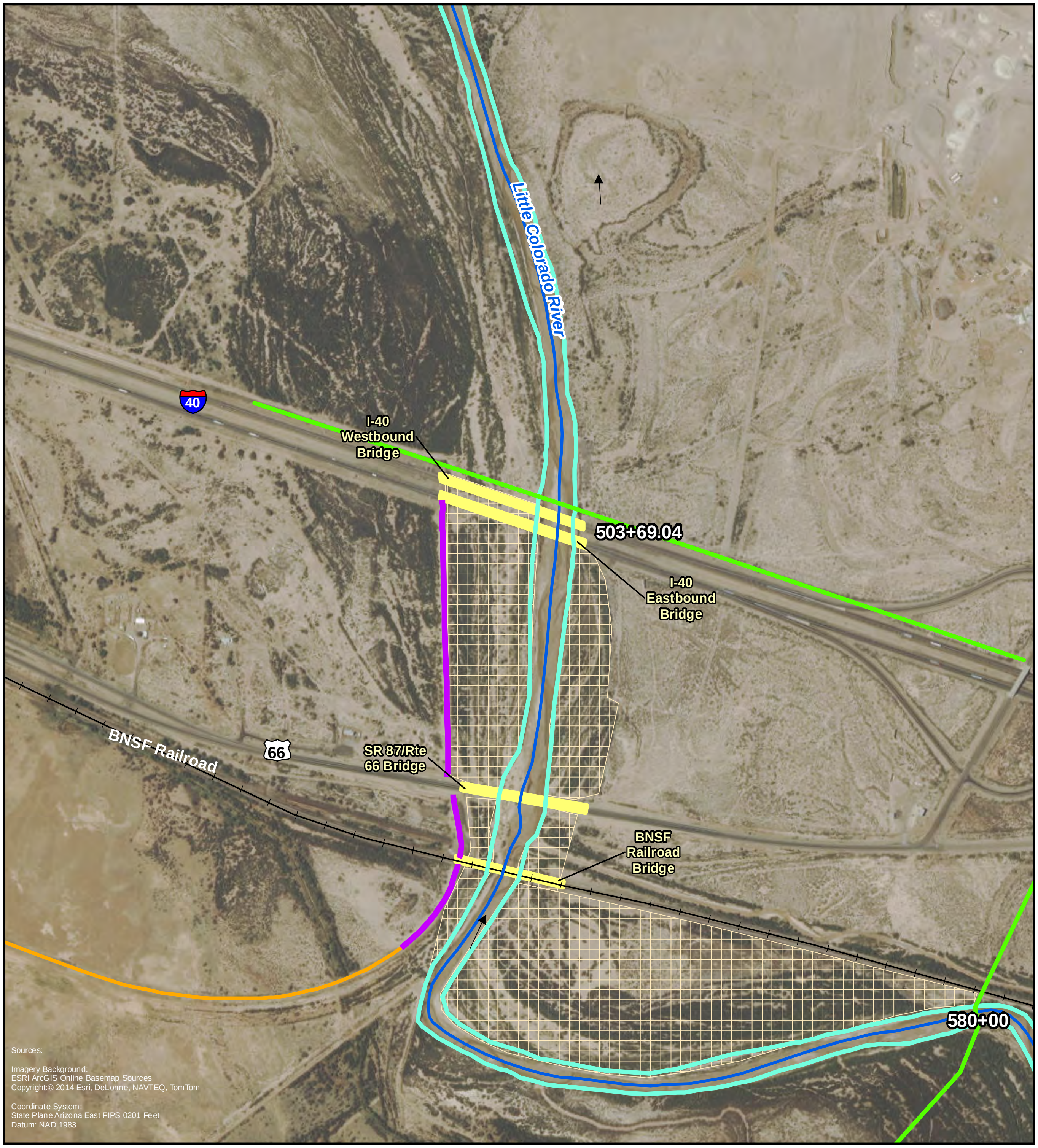
0 500 1,000 2,000 Feet

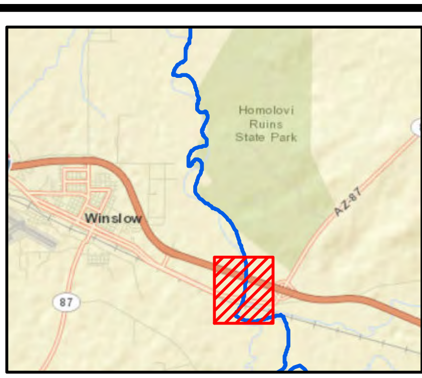
1 in = 1,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

MEASURE C
EXCAVATE & WIDEN CHANNEL
STATION 540+00 TO 480+00
(EARTHEN CHANNEL)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT





Legend

- Bounding Cross-Sections
- Little Colorado River
- Banks (Existing)
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad
- Bridges
- Remove Saltcedar

N

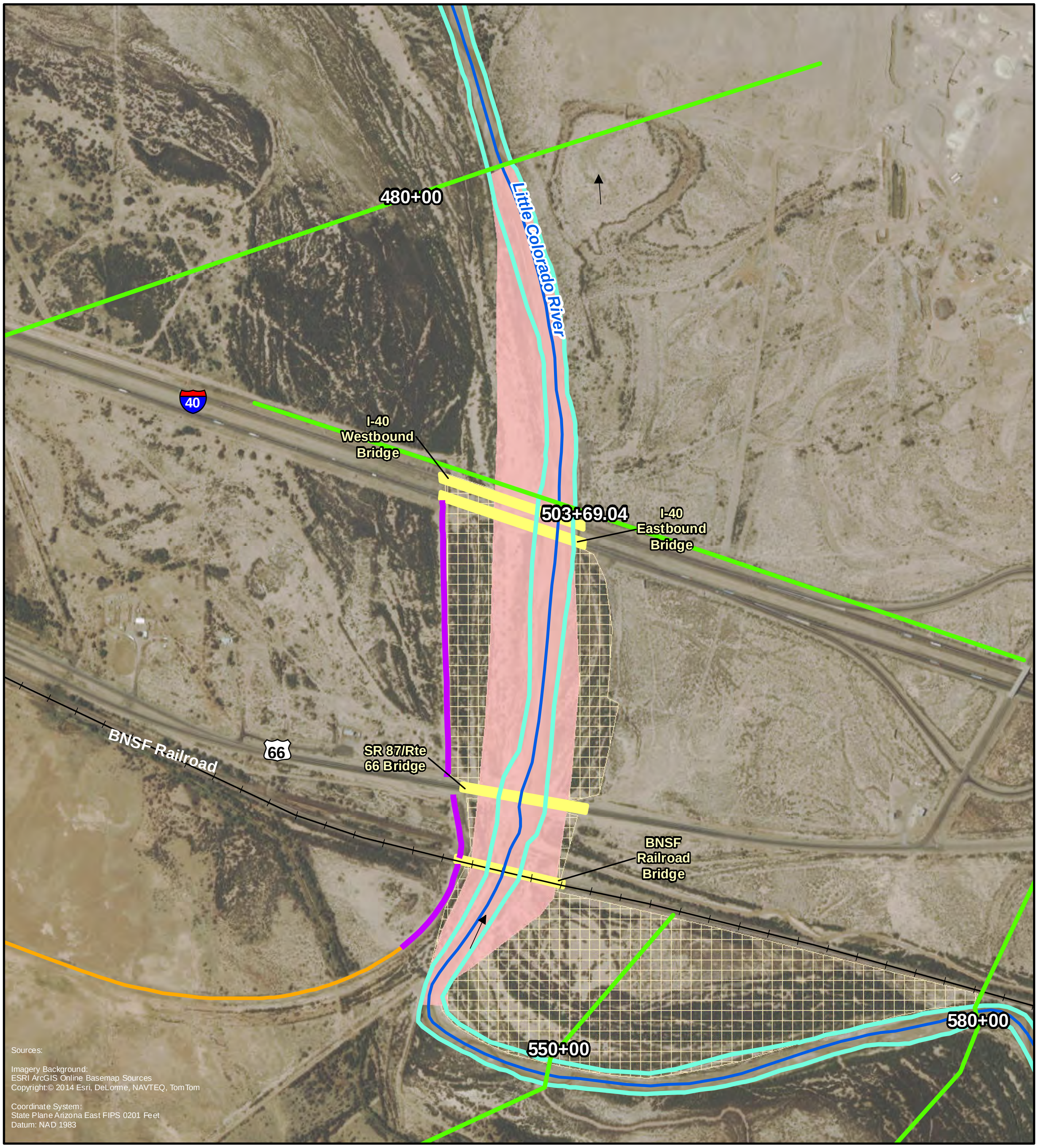
0 500 1,000 2,000 Feet

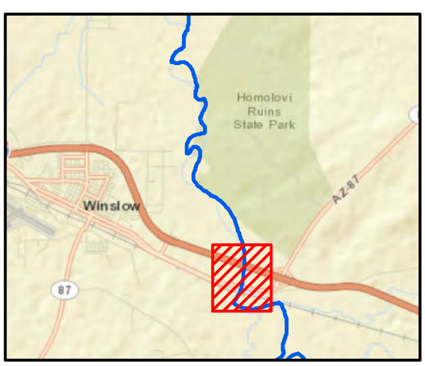
1 in = 1,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

MEASURE D
REMOVE SALTCEDAR
STATION 570+00 TO 503+69

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT





Legend

- Bounding Cross-Sections
- Little Colorado River
- Banks (Existing)
- Winslow Levee (Existing)
- RWDL (Existing)
- BNSF Railroad
- Bridges
- Excavation Area Measure E
- Remove Saltcedar

N

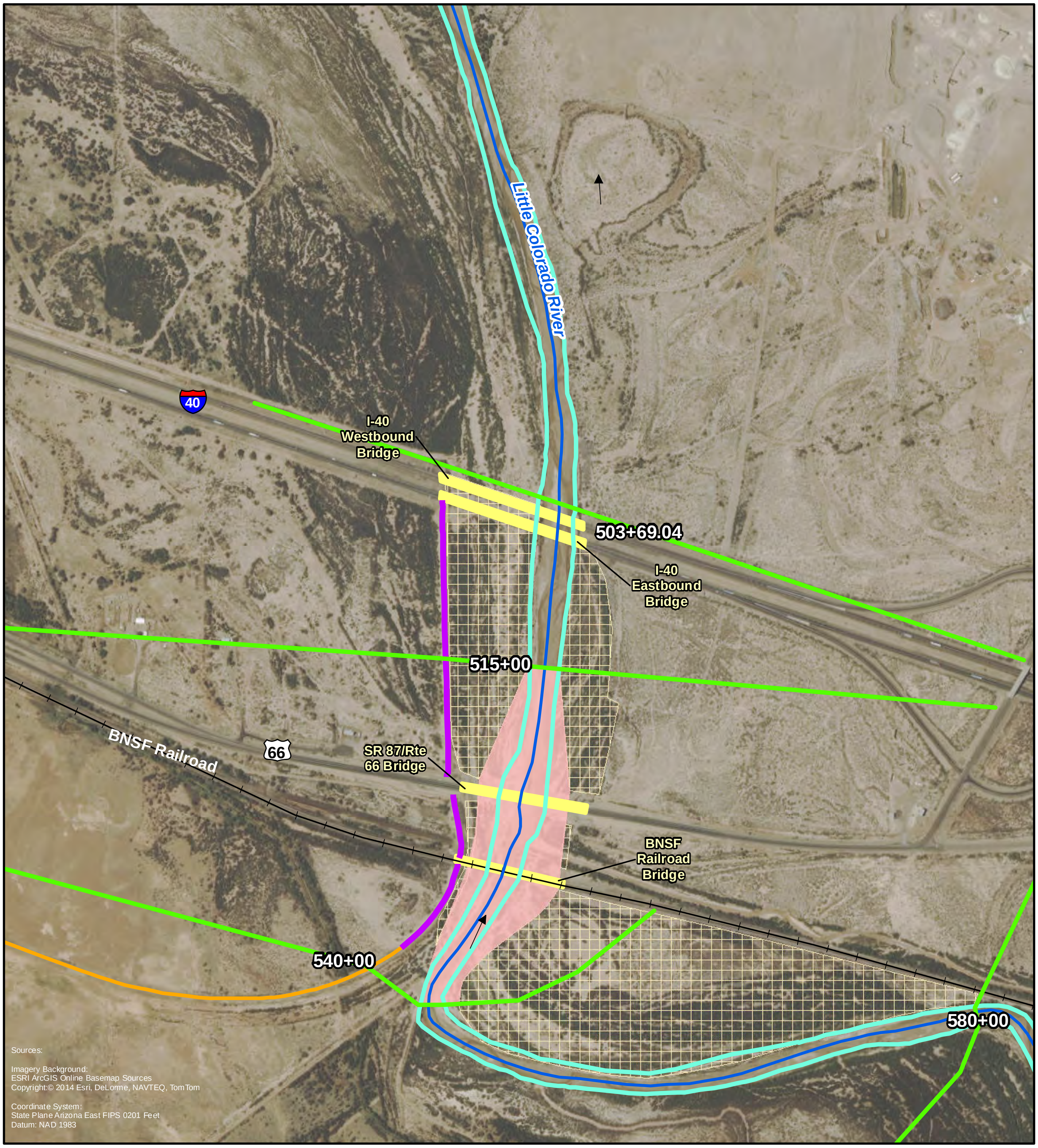
0 500 1,000 2,000 Feet

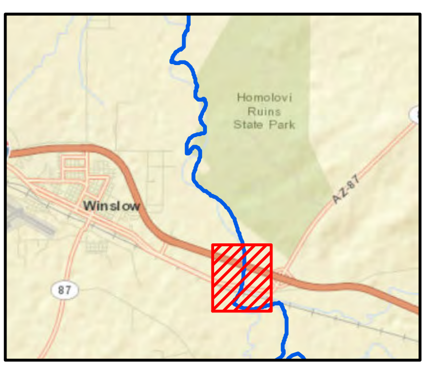
1 in = 1,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

CONVEYANCE MEASURE E
EXCAVATE & WIDEN CHANNEL
STATION 540+00 TO 480+00
(CONCRETE-LINED)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT





Legend

Bounding Cross-Sections		BNSF Railroad	
Little Colorado River		Bridges	
Banks (Existing)		Excavation Area Measure F	
Winslow Levee (Existing)		Remove Saltcedar	
RWDL (Existing)			

N

0 500 1,000 2,000 Feet

1 in = 1,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

CONVEYANCE MEASURE F
EXCAVATE & WIDEN CHANNEL
STATION 540+00 TO 515+00
(CONCRETE-LINED)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

ATTACHMENT 6

Hydraulic Analysis at Homolovi I Pueblo including Alternative 10.1 and Alternative 10.4 (Memorandum)

LITTLE COLORADO RIVER AT WINSLOW
HYDRAULIC AND SEDIMENTATION APPENDIX
APRIL 2016

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CESPL-ED-HH 17 November 2015

MEMORANDUM FOR CESPL-PM-C, ATTN: Brian Kenny

Subject: Little Colorado River at Winslow Feasibility Study – Hydraulic Analysis at Homolovi I Pueblo including Alternative 10.4

1. Purpose: The purpose of this memorandum is to present the results from the hydraulic analysis completed Alternative 10.4 relating to the Homolovi I Pueblo.

2. Background: The U.S. Army Corps of Engineers, Los Angeles District (USACE), is currently conducting the Little Colorado River (LCR) at Winslow Flood Risk Management Feasibility Study. This study is a cost-shared effort between the USACE and the Navajo County Flood Control District. The USACE Hydraulics Section was asked by Project Manager Brain Kenny to provide a summary of the hydraulic analyses completed for the LCR at Winslow Study with an emphasis at the Homolovi I Pueblo which is an archeological site along the eastern bank of the LCR on the river side of the Winslow Levee. The Homolovi I Pueblo is approximately 2 miles downstream from the Interstate 40 Bridges near Winslow. See Enclosure 1 for the location within the study area. The Homolovi I Pueblo is within the baseline conditions 1% Annual Chance Exceedance (ACE) flood event. The term “baseline condition” refers to the LCR floodplain as it exists today, in the absence of federal action to reduce the flood risk to Winslow and vicinity.

Baseline Condition Analysis

3. Baseline condition hydraulic analysis for the LCR was conducted using Hydrologic Engineering Center River Analysis System (HEC-RAS) computer software. Water surface profiles were computed and floodplain mapping was completed for the 50-, 20-, 10-, 4-, 2-, 1-, 0.5- and 0.2% ACE floods. The 1% ACE flood has a 1 in 100 chance of being equaled or exceeded in any 1 year, and it has an average recurrence interval of 100 years. It often is referred to as the “100-year flood”.

4. Scientists and engineers frequently use statistical probability (chance) to put a context to floods and their occurrence. If the probability of a particular flood magnitude being equaled or exceeded is known, then risk can be assessed. To determine these probabilities all the annual peak streamflow values measured at a streamgage are examined. A streamgage is a location on a river where the height of the water and the quantity of flow (streamflow) are recorded. For the baseline condition hydraulic model, the LCR, the Winslow Levee, and the Ruby Wash Diversion Levee (RWDL) were modeled in their existing condition. The baseline condition analysis shows that flooding begins at approximately the 10% ACE flood at the Homolovi I Pueblo site.

Alternatives Analysis Overview

5. The main objectives of the study are to reduce risks to public safety and public health for the Winslow community due to flooding and to reduce the risk of damages due to flooding in the City of Winslow and surrounding areas. A key planning objective for the study is to minimize adverse flooding and erosion impacts to the Homolovi I Pueblo. Ten action alternatives were fully evaluated by the project delivery team. Alternative 10.1 has been selected as the TSP.

Alternative 10.1 was selected as the TSP because it is the preferred alternative by the non-federal sponsor and because it complies with the requirements for the Categorical Exemption to the National Economic Development (NED) Plan which is described in the Planning Guidance Notebook *ER 1105-2-100, Chapter 3, Paragraph 3-3b(11)*. The ten action alternatives are discussed in the *LCR at Winslow Feasibility Study Baseline Condition and Alternatives Analysis Hydraulic and Sedimentation Appendix* dated August 2015.

Hydraulic Analysis

6. Alternative 10.1 includes rebuilding the Winslow Levee from the RWDL downstream to a point 0.8 miles north of North Road. It does not include improvements to the Winslow Levee downstream of station 320+00. This alternative includes setting back a short segment of the Winslow Levee across the LCR from the Homolovi I Pueblo as well as removing the original Winslow Levee in the setback area. It includes rebuilding the eastern end of the RWDL, constructing a new levee parallel to I-40, and improving conveyance under the BNSF Railroad Bridge. See Enclosure 1 for a map showing the locations of the features listed above. Alternative 10.1 was based on the water surface profiles for the 1% ACE flood.

7. Alternative 10.4 includes the same improvements as Alternative 10.1; however, Alternative 10.4 includes increased conveyance under the BNSF Railroad Bridge designed for the 0.5% ACE Flood and is based in the water surface profiles for the 0.5% ACE flood. See Enclosure 2 for a map showing the Alternative 10.4 features.

8. The other eight alternatives (Alternatives 1.1, 3.1, 7, 8, 9, 10, 10.2, and 10.3) have lower water surface elevations than the TSP and Alternative 10.4, and the eight alternatives do not increase the floodplain extents at the Homolovi I Pueblo. Further discussion can be found in the August 2015 Hydraulic and Sedimentation Appendix.

Comparison of Tentatively Selected Plan to Baseline Condition

9. A primary flood risk reduction measure for Alternatives 10.1 and 10.4 includes setting back a segment of the Winslow Levee across from the Homolovi I Pueblo as shown in Enclosures 1 and 2. The setback is approximately 1,600 feet in length. The purpose of the setback levee is to reduce the probability that the LCR will undercut the levee, which could result in a levee failure and flooding of Winslow. Setting back the levee increases flow conveyance for larger flood events (including the 1% ACE flood) near the Homolovi I Pueblo site. By increasing the flow conveyance along this reach, the water surface elevation will decrease compared to the baseline condition. See Enclosure 3 for a map showing the 1% ACE floodplain near the Homolovi I Pueblo for the baseline condition and Alternative 10.1.

10. The 1% ACE flood, which has a discharge of 69,200 cubic feet per second (cfs) along the LCR near Winslow, was chosen as the design event for the TSP. The water surface elevation at the Homolovi I Pueblo is decreased for the TSP compared to the baseline condition 1% ACE flood as a result of the setback levee. The water surface elevation decreases by approximately 0.2

feet (4852.4 feet for the baseline condition 1% and 4852.2 feet for Alternative 10.1) near the Homolovi I Pueblo. The average flow velocity in the reach near the Homolovi I Pueblo slightly decreases 0.5 feet per second (ft/s) from 4.2 ft/s from the baseline condition to 3.7 ft/s for the TSP due to the increase in conveyance and the setback levee. The decrease in average flow velocity in the reach by Homolovi I Pueblo is minimal and does not result in an increase in flooding at Homolovi I Pueblo. See Enclosure 4 for a map showing the 1% ACE floodplains near the Homolovi I Pueblo.

11. The 0.5% ACE flood, which has a discharge of 90,660 cfs along the LCR near Winslow, was chosen as the design event for the Alternative 10.4. The water surface elevation at the Homolovi I Pueblo decreases for Alternative 10.4 compared to the baseline condition (0.5% ACE flood). The water surface elevation increases by approximately 0.2 feet (4854 feet for the baseline condition 0.5% ACE flood and 4853.8 feet for Alternative 10.4) near the Homolovi I Pueblo. The average flow velocity in the reach near the Homolovi I Pueblo slightly decreases approximately 0.7 ft/s from 4.8 ft/s for the baseline condition to 4.1 ft/s for Alternative 10.4 due to the increase in conveyance and the setback levee. See Enclosure 5 for a map showing the 0.5% ACE floodplains near the Homolovi I Pueblo.

12. Hydraulic analysis shows that the proposed project (TSP) will not increase flooding at the Homolovi I Pueblo compared to the baseline (existing) condition 1% ACE flood. A portion of the Homolovi I Pueblo footprint is currently within the 1% ACE floodplain for the baseline condition. The TSP results in a decreased floodplain extent and a decreased water surface elevation at Homolovi I Pueblo compared to the baseline condition. The TSP has the same flood duration as the baseline condition. Implementation of the TSP would slightly decrease the footprint of the floodplain at the Homolovi I Pueblo due to the decrease in water surface elevation. See Enclosure 4 for a map comparing the baseline condition 1% ACE floodplain to the TSP floodplain at the Homolovi I Pueblo.

13. Hydraulic analysis shows that Alternative 10.4 would slightly decrease flooding at the Homolovi I Pueblo compared to the baseline condition 0.5% ACE flood. A portion of the Homolovi I Pueblo footprint is currently within the 0.5% ACE floodplain for the baseline condition. Alternative 10.4 has the same flood duration as the baseline condition. See Enclosure 5 for a map comparing the baseline condition 0.5% ACE floodplain to Alternative 10.4 0.5% ACE floodplain at the Homolovi I Pueblo.

14. Please contact Adam Bier at 213-452-3567 to discuss any questions regarding the analysis.

Encls

Adam J. Bier, P.E
Senior Hydraulic Engineer
Hydraulics Section

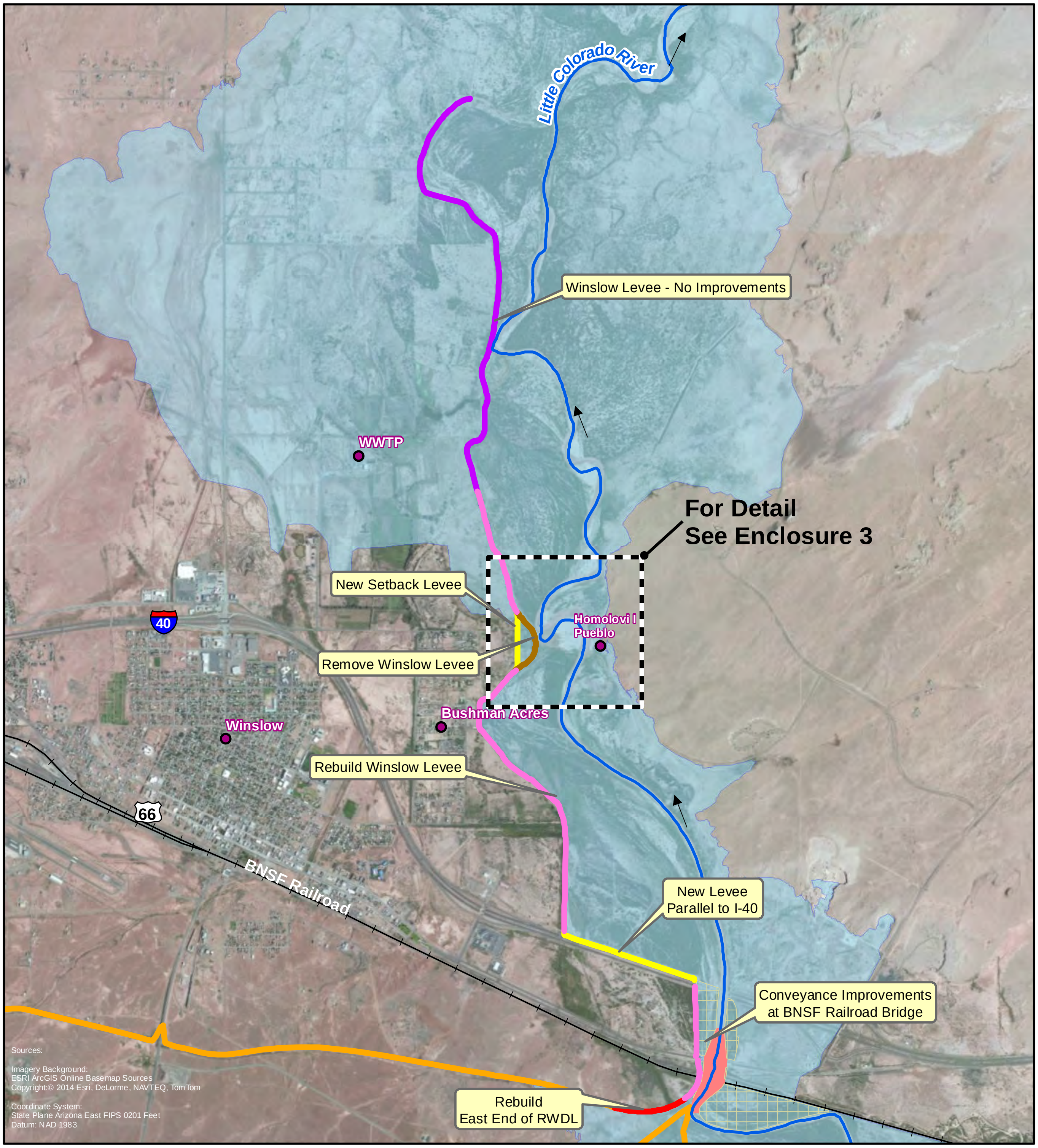
CESPL-ED-HH
Subject: Little Colorado River at Winslow Feasibility Study – Hydraulic Analysis at Homolovi I Pueblo including Alternative 10.4

CF: CESPL-PM-C (Kenny)
CESPL-PD-WC (Legere)
CESPL-ED-HH

RV 12-3-15
VERMEEREN
CESPL-ED-H

MRSE 3-DEC-15
CESPL-ED-HH

BIER AB 12/3/15
CESPL-ED-HH



Legend

- Little Colorado River
- Rebuild Winslow Levee
- Remove Winslow Levee
- New Levee
- Winslow Levee - No Improvements
- Rebuild RWDL
- RWDL - No Improvements
- Remove Saltcedar
- Conveyance Improvements
- BNSF Railroad
- TSP 1% ACE Floodplain

N

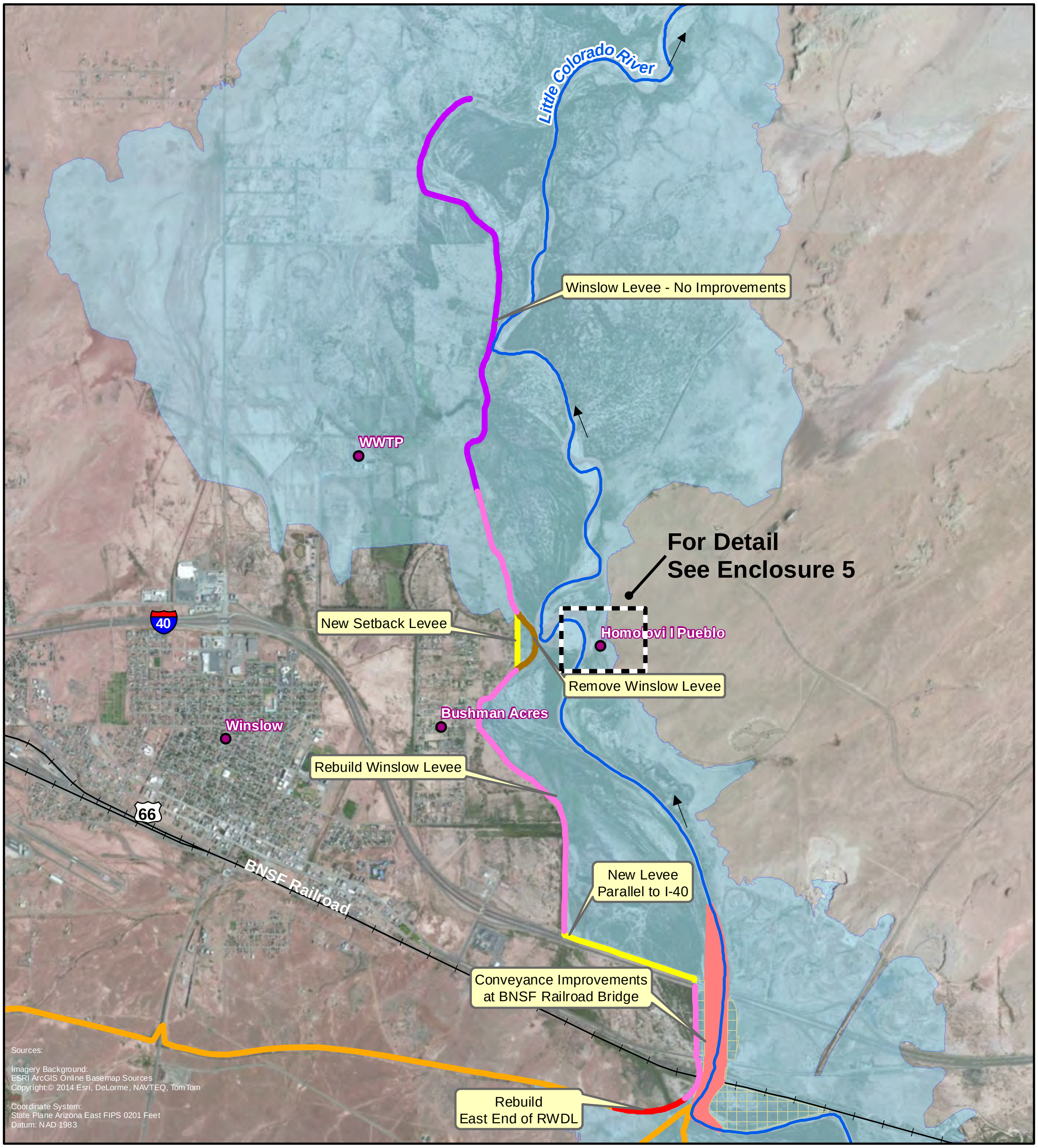
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 10.1
REBUILD EAST END OF RWDL
REBUILD & SETBACK LEVEES
NEW LEVEE ALONG I-40
CONVEYANCE IMPROVEMENTS
BASED ON 1% ACE FLOOD

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Little Colorado River
- Rebuild Winslow Levee
- Remove Winslow Levee
- New Levee
- Winslow Levee - No Improvements
- Rebuild RWDL
- RWDL - No Improvements
- Remove Saltcedar
- Conveyance Improvements
- BNSF Railroad
- Alternative 10.4 - 0.5% ACE Floodplain

N

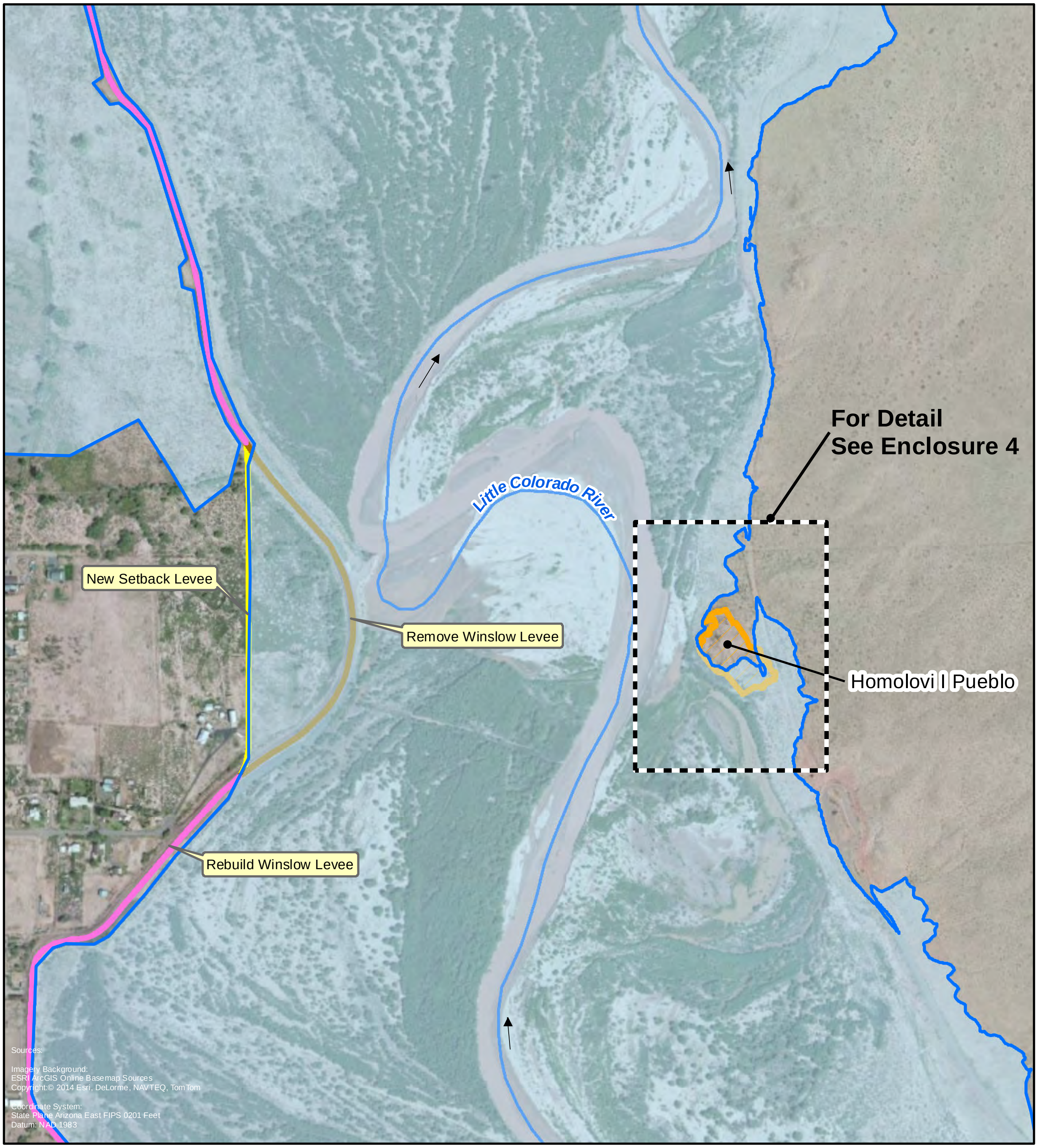
0 2,000 4,000 8,000 Feet

1 in = 4,000 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

ALTERNATIVE 10.4
REBUILD EAST END OF RWDL
REBUILD & SETBACK LEVEES
NEW LEVEE ALONG I-40
CONVEYANCE IMPROVEMENTS
BASED ON 0.5% ACE FLOOD

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- TSP - 1% ACE Floodplain
- Homolovi I Pueblo Footprint
- Little Colorado River
- Rebuild Winslow Levee
- Remove Winslow Levee
- New Levee

N

0 350 700 1,400 Feet

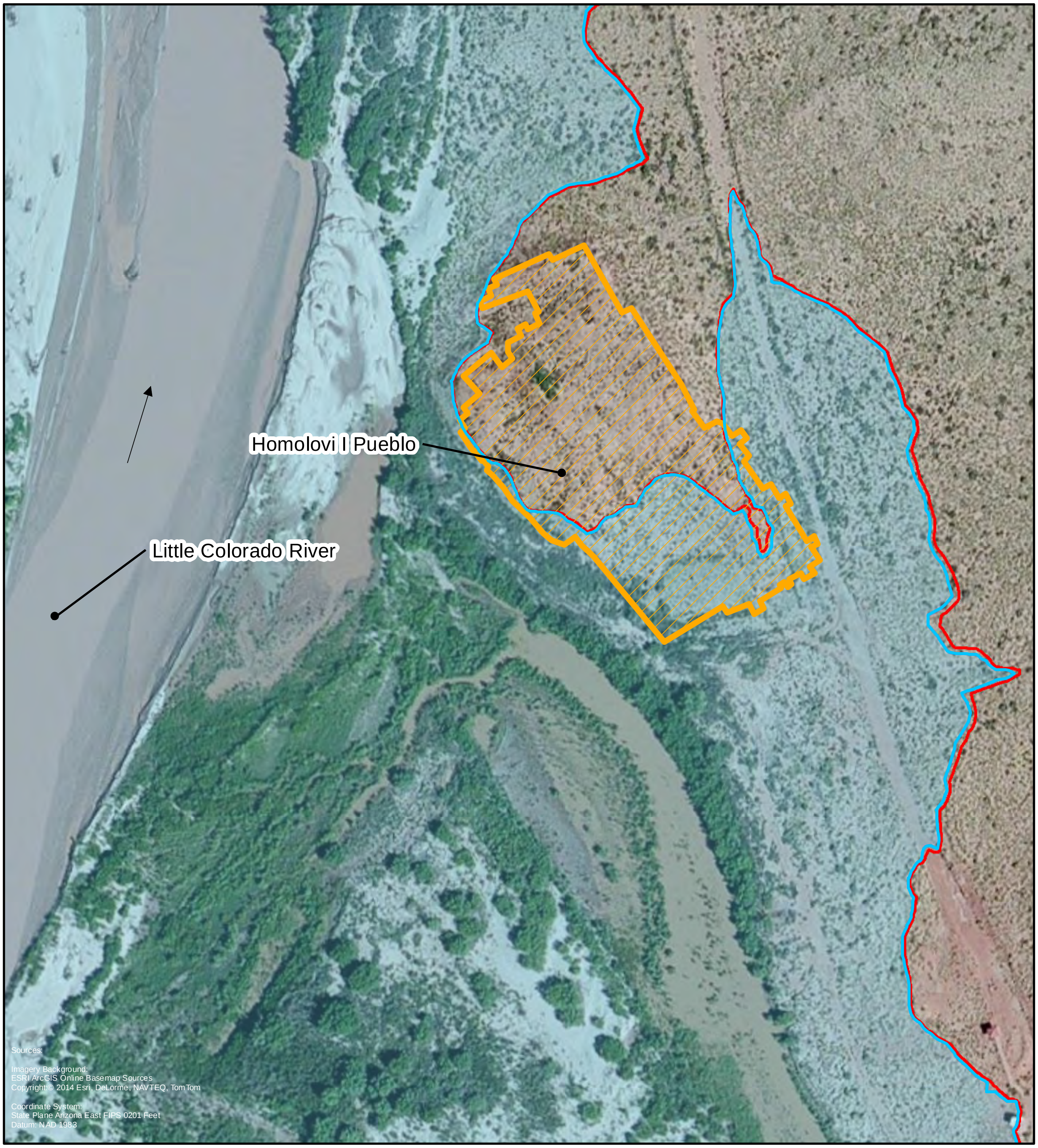
1 in = 700 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

TENTATIVELY SELECTED PLAN
FLOODPLAIN NEAR
HOMOLOVI I PUEBLO
1 % ACE FLOOD

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

ENCLOSURE 3



Legend

- Homolovi I Pueblo Footprint
- Baseline - 1% ACE Floodplain
- TSP - 1% ACE Floodplain

N

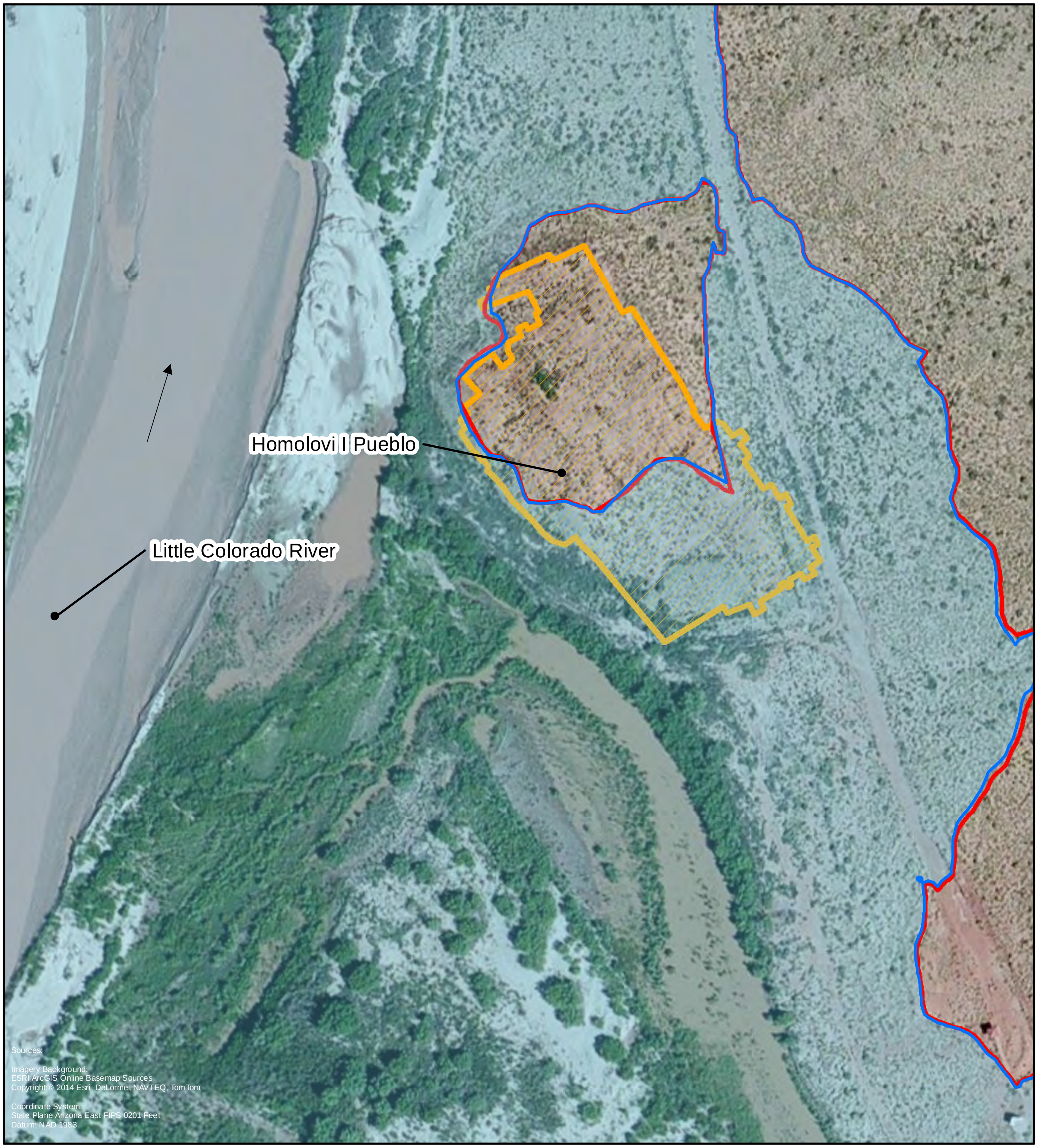
0 75 150 300 Feet

1 in = 150 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

COMPARISON OF
TENTATIVELY SELECTED PLAN
AND BASELINE CONDITION
HOMOLOVI I PUEBLO
(1% ACE FLOOD)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



Legend

- Alternative 10.4 - 0.5% ACE Floodplain
- Baseline - 0.5% ACE Floodplain
- Homolovi I Pueblo Footprint

N

0 75 150 300 Feet

1 in = 150 feet

LITTLE COLORADO RIVER
AT WINSLOW
FEASIBILITY STUDY WINSLOW, AZ

COMPARISON OF
ALTERNATIVE 10.4
AND BASELINE CONDITION
HOMOLOVI I PUEBLO
(0.5% ACE FLOOD)

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

ENCLOSURE 5